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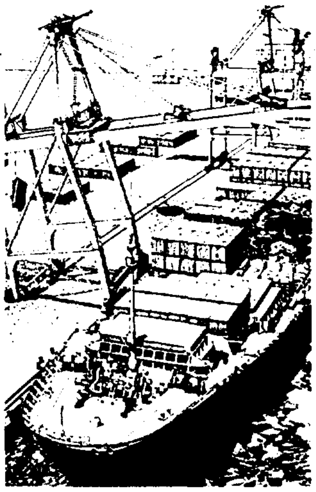

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TECHNICAL REPORT GL-88-16

EVALUATION AND REPAIR OF WAR-DAMAGED PORT FACILITIES

Report 2

PORT VULNERABILITY, PIER AND WHARF REPAIR AND STORAGE AREA REPAIR

by

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US ARMY ENGINEER WATERWAYS EXPERIMENT STATION

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The difference between victory and defeat during military confrontations depends significantly on a nation's ability to establish and maintain military supply lines between the home front and the theater of operations. One such supply line is the transfer of cargo from ships, across waterfront facilities, and inland. Since large quantities of cargo are delivered by sea, it is imperative that military supplies be moved through ports and into the theater.

Port facilities are potential targets for hostile forces. Ports can be expected to be attacked to render the facilities inoperative or to deny access to the facilities. Port repairs should be conducted as quickly as possible to restore war-damaged areas for the transfer of military supplies from support ships to shore facilities and inland.

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19. ABSTRACT (Continued). → This report presents

The studies reported herein present information and repair concepts for piers, wharves, storage areas, and hardstands that will ensure continuing port operations during emergencies caused by military conflicts. A port vulnerability analysis was conducted on a selected commercial and military port, and predictions were made to port facilities on damage resulting from an aerial general purpose bomb threat. Cast-in-place concrete, prefabricated concrete, and military bridging are identified and discussed as possible repairs for war-damaged piers and wharves. The span capability of aluminum extrusions was traffic tested and evaluated. ~~as reported herein.~~ The extrusion repair concept has the potential for repairing bomb-damaged pier and wharf decking. All airfield damage repairs were cataloged and analyzed, and applicable techniques and materials are presented as repairs to bomb-damaged storage areas and other facility pavements at ports. Key[illegible]
Waterfront structures; Bomb damage; [illegible] operations;

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PREFACE

The investigation reported herein was sponsored by the US Army Corps of Engineers (USACE) and was conducted under Project AT40, Task CO, Work Unit 009, "Evaluation and Repair of War-Damaged Port Facilities." Mr. Michael Shama was Technical Monitor for USACE.

The study was conducted at the US Army Engineer Waterways Experiment Station (WES) from January 1983 to September 1986 by the Pavement Systems Division (PSD) of the Geotechnical Laboratory (GL) and Structural Mechanics Division (SMD) and Concrete Technology Division (CTD) of the Structures Laboratory (SL). The work was conducted under the general supervision of Dr. W. F. Marcuson III, Chief, GL, WES, Mr. B. Mather, Chief, SL, WES, and Dr. J. P. Balsara, Chief, SMD, SL. Mr. R. E. Walker, SMD, provided technical supervision. Direct supervision was provided by Messrs. H. H. Ulery, Jr., Chief, PSD, GL, and J. M. Scanlon, Chief, CTD, SL. Personnel of the PSD involved in this study were Messrs. H. L. Green, R. H. Grau, and D. M. Ladd. This study was coordinated by Mr. C. J. Smith, PSD, who was the Principal Investigator.

The port vulnerability study (Part II) was prepared by Ms. F. M. Warren, SMD, SL. Mr. E. F. O'Neil, CTD, SL, prepared Part III and supporting appendixes for pier and wharf repair using concrete and military bridging.

Repair of war-damaged storage areas and other facility pavements at ports (Part V and supporting appendix) were prepared by Mr. D. L. Cooksey, PSD. Parts I, IV, and VI were prepared by Mr. Smith, PSD, who was also responsible for assembling the final report. This report was edited by Ms. Odell F. Allen, Information Products Division, Information Technology Laboratory.

Commander and Director of WES was COL Dwayne G. Lee, EN. Technical Director was Dr. Robert W. Whalin.

CONTENTS

CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
cubic feet	0.02831685	cubic metres
cubic yard	0.7645549	cubic metres
degrees (angle)	0.01745329	radians
Fahrenheit degrees	5/9	Celsius degrees or Kelvins*
feet	0.3048	metres
foot-pounds (force)	1.355818	metre-newtons or joules
gallons	3.785412	cubic decimetres
gallons per square yard	4.5273	cubic decimetres per square metre
inches	2.54	centimetres
kips (force)	4.448222	kilonewtons
miles (US statue)	1.609347	kilometres
pounds (force) per foot	14.5939	newtons per metre
pounds (force) per square foot	47.88026	pascals
pounds (force) per square inch	6.894757	kilopascals
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic inch	27.6799	grams per cubic centimetre
square feet	0.09290304	square metres
square inches	6.4516	square centimetres
tons (2,000 pounds, mass)	907.1847	kilograms
yards	0.9144	metres

* To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use the following formula: $C = (5/9)(F - 32)$. To obtain Kelvin (K) readings, use: $K = (5/9)(F - 32) + 273.15$.

EVALUATION AND REPAIR OF WAR-DAMAGED PORT FACILITIES

Report 2

PORT VULNERABILITY, PIER AND WHARF REPAIR, AND STORAGE AREA REPAIR

PART I: INTRODUCTION

Purpose and Scope

1. This study provides guidance and information to ensure continued port operations during emergencies caused by military confrontations. Specific purposes were to develop and evaluate materials and systems for expedient repair of port facilities, and to develop design/construction methods for rapid rehabilitation of port facilities. Results of this study will enable US Armed Forces and/or indigenous personnel to restore port areas that permit access for transferring supplies from support ships to shore facilities and inland.

2. This investigation includes a port vulnerability study to determine the anticipated damages to piers/wharves and storage areas. The vulnerability analysis was conducted on a selected commercial and military port to predict the damage (number of holes and structural damage) resulting from an aerial general purpose bomb threat. Office studies were conducted to identify expedient repair methods, materials, and equipment for war-damaged piers/wharves, storage areas, and hardstands. Although concrete construction is not considered an expedient repair, cast-in-place and precast concrete procedures are included as repair techniques for piers/wharves. Aluminum extrusions were evaluated to determine their span capability for repair of bomb-damaged pier/wharf decking. The extrusions were trafficked with wheel-loadings simulating container handling port traffic and evaluated on their performance to span various-sized holes without structural failure.

3. This is the second of four reports on the subject work unit. Report 1 is a Waterways Experiment Station (WES) study which identifies port construction in previous military conflicts, provides information for war-damaged port assessment, and presents compendiums of major ports with special characteristics. The Naval Civil Engineering Laboratory (NCEL) has conducted

studies to identify expedient repair methods, designs, and materials for war-damaged pier/wharf decking and supporting piling. NCEL results are presented in Reports 3 and 4. Report 3 focuses on concepts and design solutions that can be used for the expedient repair to pier/wharf decking. Report 4 presents concepts for the expedient underwater repair of piers/wharves and quay walls.

Port Targets

4. In an all-out conflict, hostile action can be anticipated against targets in the combat and communication zones which include the reinforcement and resupply ports of entry into the theater. Bombings, sabotage, and chemical attacks are all possibilities and may be directed at the area immediately behind the combat zone to obstruct the movement of reinforcements through ports and into the theater.

5. Port losses in terms of facilities, equipment, and personnel can be expected in a postattack environment. Hostile action is anticipated to be conducted against the transportation networks (shipping channels, locks and bridges, piers/wharves, materials-handling equipment, storage areas, roadways, railways, etc.), communication networks, military depots, power supplies supporting civilian facilities, and any target where action would degrade the ability of forces to fight effectively. Anticipated port damage due to military action is depicted in an artist's concept (Figure 1).

Ordnance Removal

6. The removal of unexpended ordnance is an extremely hazardous operation and should be attempted only by trained Explosive Ordnance Demolitions (EOD) personnel. A complete survey by EOD personnel is recommended prior to beginning any damage assessment or rubble removal. If unexpended ordnance is located after work has begun, all personnel should clear the area and await the specialized assistance of EOD personnel. On occasion, EOD personnel may not be available, and repairs must proceed. If that is the case, damage assessment and rubble removal should take place very carefully, and if unexpended ordnance is located, it should be destroyed in place by use of controlled blasting. The details of ordnance disposal are presented in FM 9-15 (Headquarters, Department of the Army 1984), TM 9-1375-213-12

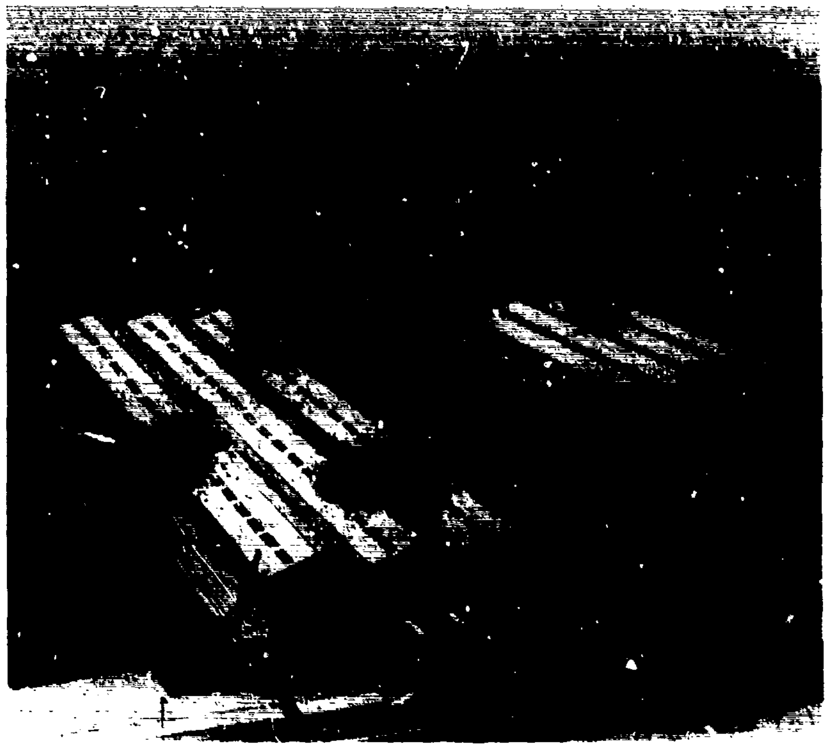

Figure 1. Artist's concept of a war-damaged port

(Headquarters, Department of the Army 1973a), and TM 9-1300-206 (Headquarters, Department of the Army 1973b).

Damage Assessment

7. The movement of supplies is currently anticipated through major and selected secondary ports. These ports include the receiving facilities, connecting transportation networks, and civil networks and are vital to support defense-related activities.

8. Planners can expect ports to be attacked in various ways to render the facilities inoperative or to deny access to the facilities. After an attack, priority should be given to bringing the initial effects of the attack under control. Fires should be extinguished; ships and equipment in danger of damage should be relocated, and wounded personnel should be removed and treated. An initial assessment should be made to determine whether the port can continue the discharge of ships already in the port. If shore cranes are

damaged or unavailable, alternate discharge sites or use of ship's equipment should be considered.

9. A detailed assessment of port damage should be made as soon as possible after an attack. The damage estimate should first analyze the crucial areas of a port which could cause operations to completely cease. Crucial areas include access to the port (channel blockage, lock damage, mine blockades, etc.); discharge facilities (container cranes, cargo handling equipment, berth space, etc.); and inland cargo routes (storage, railroads, roadways, etc.).

Repair Decisions

10. According to TM 5-360 (Headquarters, Department of the Army 1964), a decision to rehabilitate or abandon war-damaged port facilities should be based on: (a) inspection to determine the extent of damage, (b) the importance of the facility in relation to the overall improvement of the port facility, and (c) limitations on use of the facility after rapid repairs have been made. Various inspection techniques related to damaged port facilities, such as piers/wharves, storage areas, marshaling yards, and hardstands, are presented in Technical Report GL-86-6 (Smith 1986).

11. The information obtained by an above-water and underwater inspection can be used to make an engineering assessment of the war-damaged waterfront facility, its capacity to carry loads, and its capability to be used safely for its intended purpose for a projected period of time. After the findings of the engineering assessment are made available, a decision (to repair and what to repair, to replace and what to replace, etc.) can be made on the necessary action to take concerning the operational use of the damaged structures. Rational engineering decisions can be made on the feasibility of repairing the existing damaged port structures versus abandonment of the damaged structures and moving to an alternate port or implementing alternate discharge methods. Logistics over-the-shore (LOTS) operations provide an alternate means of resupplying theater of operations as port facilities become inoperative during military confrontations. Engineering decisions made to repair damaged port structures could include LOTS operations combined with the port rehabilitation process. LOTS operations could supply the theater while repair of port structures are being achieved.

PART II: PORT VULNERABILITY

Introduction

12. The military is dependent on supplying large volumes of cargo in the theater of operations through the use of strategic ports and their facilities. Port facilities are potential targets for hostile forces. Ports can be expected to be attacked in various ways to render the facilities inoperative or to deny access to the port facilities. The following study was conducted to evaluate the vulnerability of ports to damage by a selected threat of air-to-surface munitions. Results from the study were used as design criteria for the expedient repair of damaged port facilities.

13. Targets such as port facilities are categorized as "soft" targets. A soft target is defined as a target that is not specifically designed and constructed for the purpose of defeating weapon effects. Several attack philosophies are normally considered in a vulnerability analysis:

a. Minimum physical damage with denial of use for short time.

b. Major physical damage with long-term denial of use to the point of reconstruction.

14. This study is based on a selected military threat which uses the attack philosophy of "a", described above, and addresses the physical damage (i.e., holes and structural damage) to repair. An Airbase Damage Assessment (AIDA) computer model (Emerson 1976) was used to determine the number of holes to repair. This model was modified by WES to account for area weighted probabilities for bomb damage assessment. Input to the model includes an attack plan for each plane flown in an attack and a target layout. An attack plan is defined by the expected probability of arrival, heading, aim point, delivery accuracy, and dispersion for a stick of weapons, i.e., an aerial bomb delivery for straight line bomb placement on ground targets. All bombs released were given a probability of 1 for reliability, i.e., all bombs released will detonate. The model incorporates a Monte Carlo mode and an expected value mode to determine where the bombs land. The Monte Carlo mode uses random variables drawn from the appropriate error distributions to determine the mean point of impact and the actual impact points. The expected value mode derives an average value of the hit density for each target and for

each attack and then combines these to provide the total expected number of hits for all the attacks.

15. Even though the scope of the port project covers only structural damage, this study would be negligent to ignore other target characteristics that increase the vulnerability of the port facility. These additional "weapon effects" are pointed out as the individual targets within the port facility are discussed. The vulnerability study addresses the pertinent weapon effects for the port targets, attack scenarios, and the results of the attack simulations.

Representative Port

16. A model port was selected to conduct the vulnerability study. The representative port was based on a selection from 200 container ports in the world. Of the top 50 ports, Hampton Roads, VA, represented a mid-range generic port measured in 20-ft* equivalent units (container traffic). The generic port was also near several military installations. It contains a commercial container port and a naval base. The port has a strategic military location and data which were accessible and readily available.

Terminology

17. A pier/wharf is a term used to describe a discharge facility for unloading and loading the cargo that a vessel transports. A pier generally is a structure that projects into the water. It usually provides berthing on both sides of the structure and is perpendicular to or at an acute angle to the shore. A wharf is a facility that usually parallels the shore. The number of linear feet at the face of the wharf is the length of the berthing accommodation.

Construction types

18. There are two general types of pier/wharf construction: open construction and solid construction. Structures of open construction consist of a pile-supported deck with water below. Structures of solid construction consist of filled caissons or similar cells or sheet piling walls tied together and backfilled. A combination of solid and open construction may also be used for ship discharge facilities.

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* A table of factors for converting non-SI units of measurement to SI (metric) units is given on page 4.

Generic port facilities

19. A commercial and military port facility at Hampton Roads, VA, was selected to conduct the vulnerability analysis. Commercial port activities in Virginia are coordinated by the Virginia Port Authority with headquarters in Norfolk, VA. The port authority owns five major cargo terminals at the Port of Hampton Roads which include Norfolk International Terminal (NIT), Portsmouth Marine Terminal, Newport News Marine Terminal, Lamberts Point Docks, and Sewells Point Docks. The Port of Hampton Roads consists of older wharves constructed of creosote wood piling with concrete decking and modern container wharves constructed of concrete piling and decking. The facilities at NIT were selected as the generic commercial port (Figure 2) to conduct the target and weapon effect analyses.

Figure 2. General view of NIT

20. The openly constructed container wharf (Figure 3) was selected as the generic structure within NIT for the weapon effect analysis. The wharf consists of piles spaced at 20-ft intervals running parallel to the wharf's edge and at 11-ft intervals perpendicular to the wharf's edge (Figures 4 and 5). Structural components are composed of approximately 18-ft long by 4-ft wide precast prestressed concrete planks placed on cast-in-place concrete pile caps, and the planks are overlayed with cast-in-place concrete

Figure 3. View of container wharf at NIT

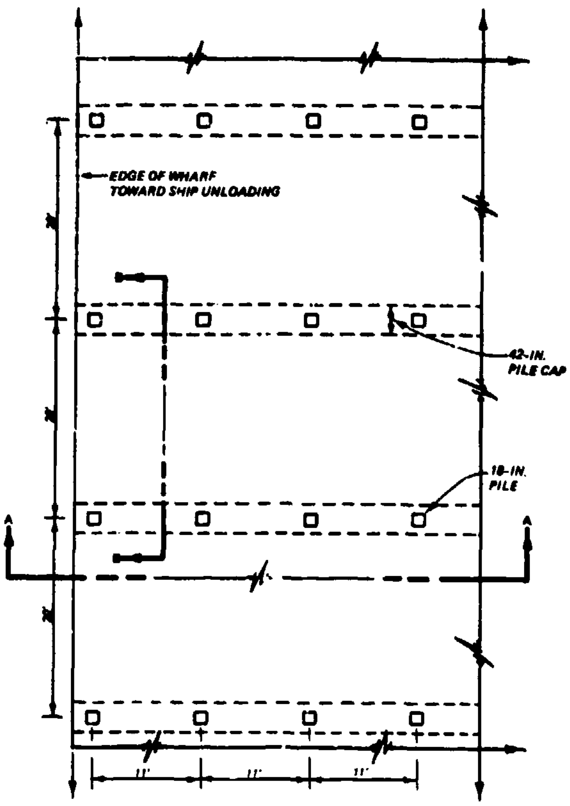


Figure 4. Plan view of container wharf at NIT

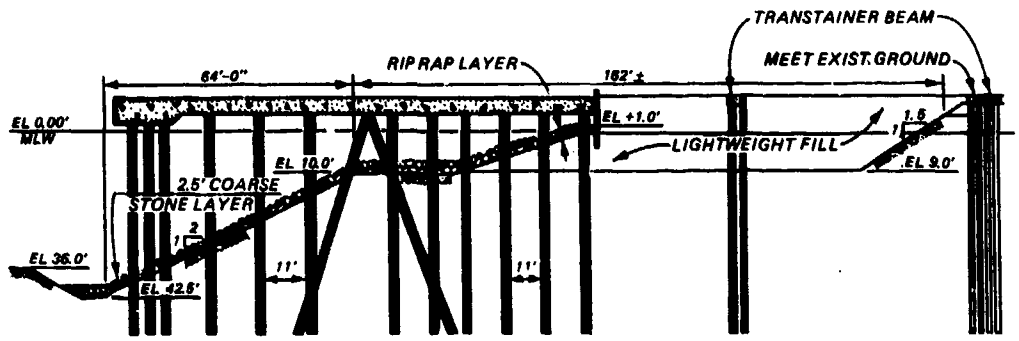


Figure 5. Section A-A of NIT container wharf

(Figure 6). The deck depth is approximately 1 ft thick. This container wharf construction represents the type of port construction currently used across the country.

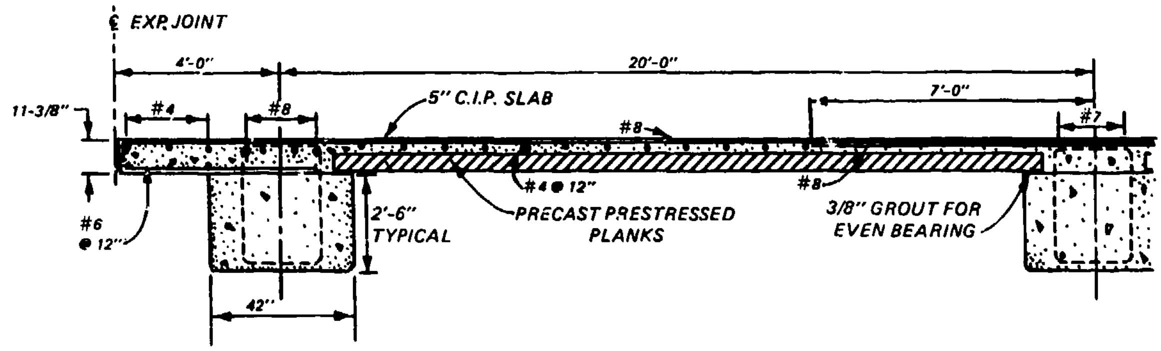


Figure 6. Section B-B of container wharf

21. The port facilities at the Norfolk Naval Base were selected as the generic military port to conduct the vulnerability analysis. The naval base port facilities include more than 13 openly constructed piers which extend into the Elizabeth River. Pier 10 represents the latest design and was selected as the generic structure for conducting the target and weapon effect analyses. Plans and design have been prepared for construction of pier 10, but the pier had not been constructed at the time of this report. Pier 10 (Figure 7) consists of piles spaced at 8-ft-9-in. intervals along the concrete cast-in-place pile caps (Figure 8a). The 18-in.-thick concrete cast-in-place decking spans 18 ft between pile caps (Figure 8b). Another similarly constructed older structure, pier 7, consists of 16-in. square concrete piles spaced at 8-ft intervals along cast-in-place concrete pile caps. The 8-in.-thick concrete decking spans 12 ft between pile caps. It should be noted that

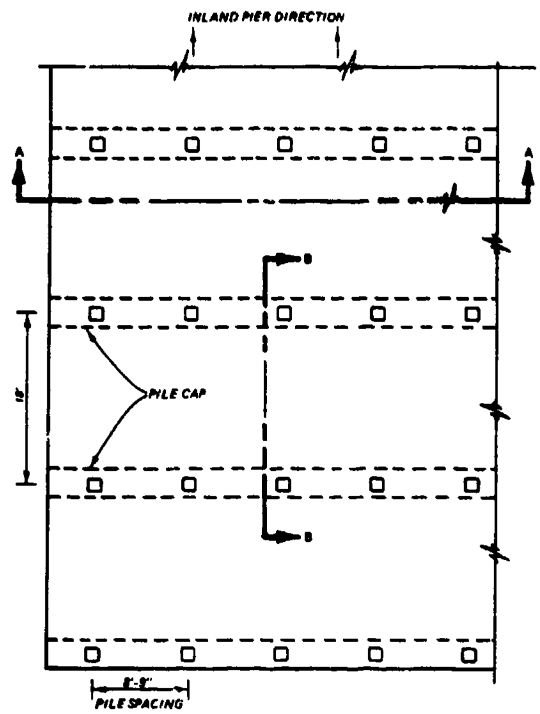
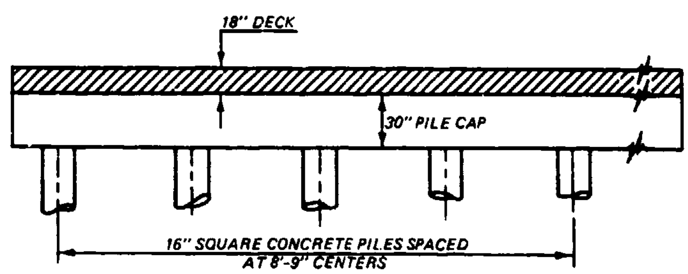
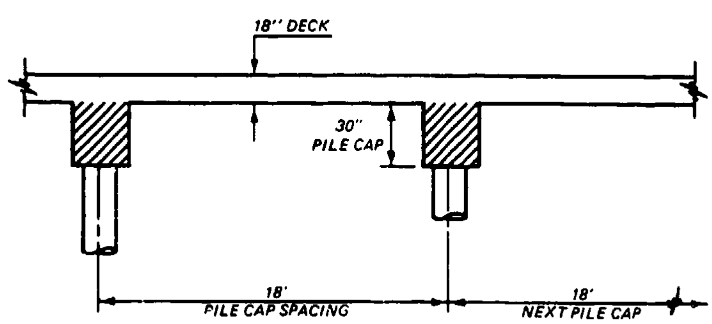


Figure 7. Plan view of pier 10



a. Transverse Section A-A



b. Longitudinal Section B-B

Figure 8. Pile and pile cap spacing of pier 10

structural design characteristics will differ according to as-built design specifications and subsurface soil conditions.

Threat and Weapon Effects

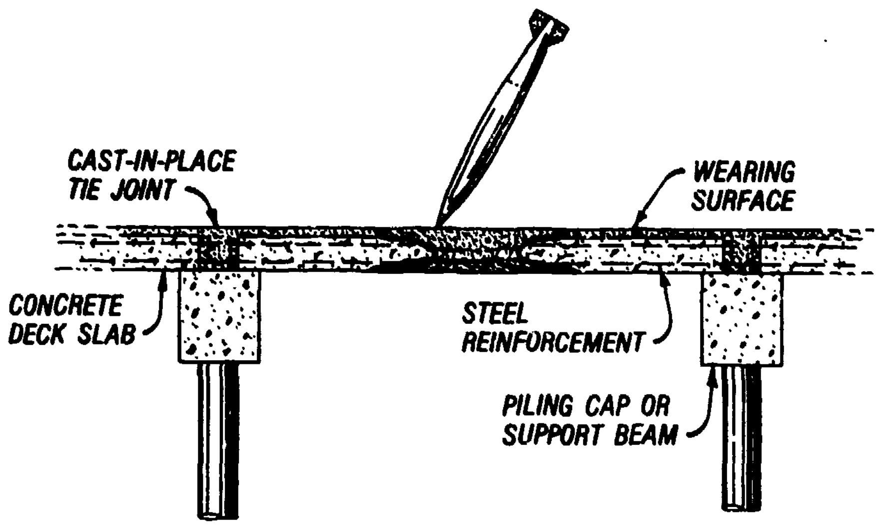
22. A general purpose 500-lb bomb (MK-82 warhead) which contains approximately 250 lb of high explosives was used in this study for the physical damage threat. This weapon was assumed to be contact fuzed for the wharf and pier targets and delay fuzed for all other port targets. These fuze selections give maximum mechanism for the current selected targets. Contact-fuzed bombs are assumed to detonate immediately upon nose impact with the target. Two potential types of damage are possible from contact bomb detonations against piers/wharves of open construction: cratering of the deck with associated spalling and structural failure of the deck slab and/or supporting beams by bending deflections (Figure 9). The potential damage mechanisms to piers/wharves from delay-fuzed bomb detonation are shown in Figures 10 and 11. Figure 12 summarizes the vulnerability of typical piers/wharves of open construction to contact- and delay-fuzed bombs. The vulnerability levels are expressed in terms of damage radii as a function of bomb detonation position. The damage mechanisms against piers/wharves are discussed in detail by Davis and McMahon (in preparation).

Pier/wharf weapon effect

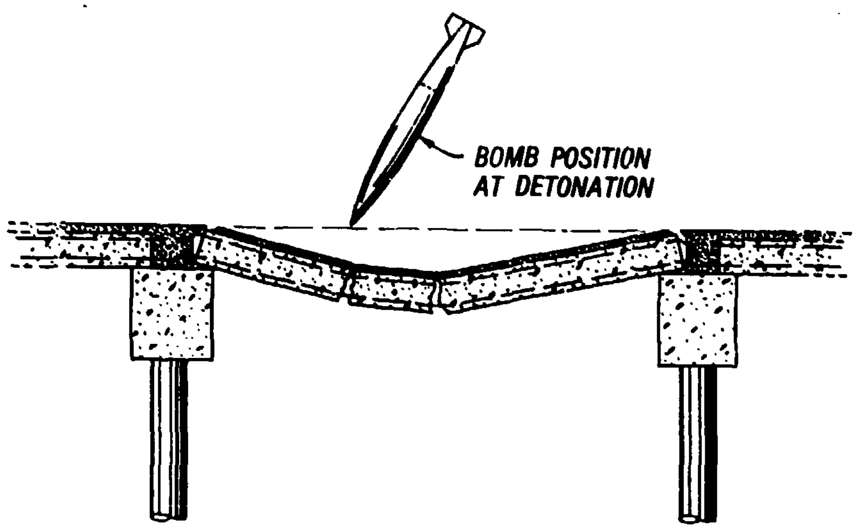
23. Weapon effects on concrete piers/wharves were analyzed for both the generic commercial and military ports. An average deck thickness of 1 ft was assumed to represent pier/wharf construction. The diameter of back craters in concrete piers/wharves resulting from a 500-lb bomb detonating on the deck surface was determined by using Figure 13 (Ball 1976). The expected crater diameter from a bomb containing 250 lb of high explosives and detonating on a 1-ft-thick concrete deck is 8.4 ft.

Storage area weapon effect

24. Container terminals at major ports consist of large container storage areas which are located adjacent to container ship berths. Containers at NIT are parked in paved storage yards on a trailer chassis awaiting road truck arrival for delivery to destination points (Figure 14). Other containers are stored and stacked three containers high on paved surfaces using rail mounted cranes (Figure 15). Container storage areas consist of the following

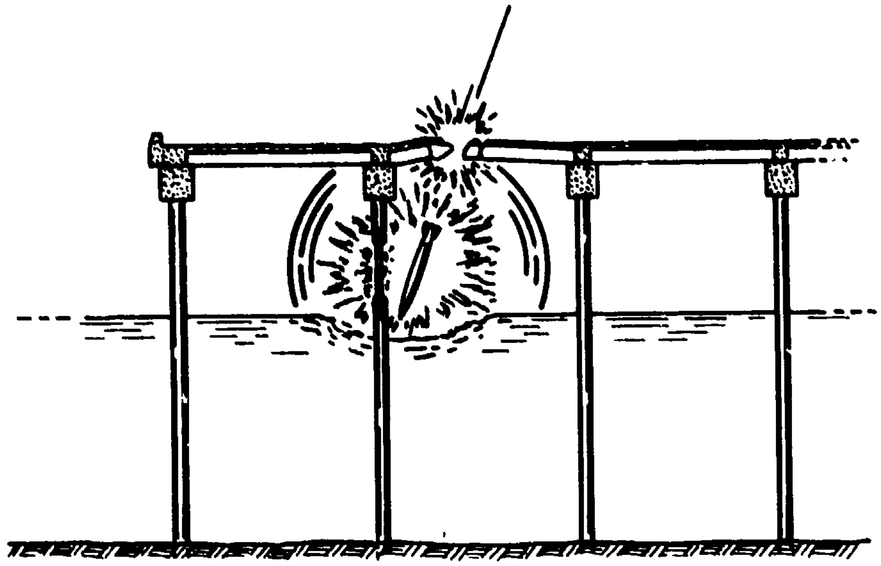


a. Deck perforation by cratering and spalling

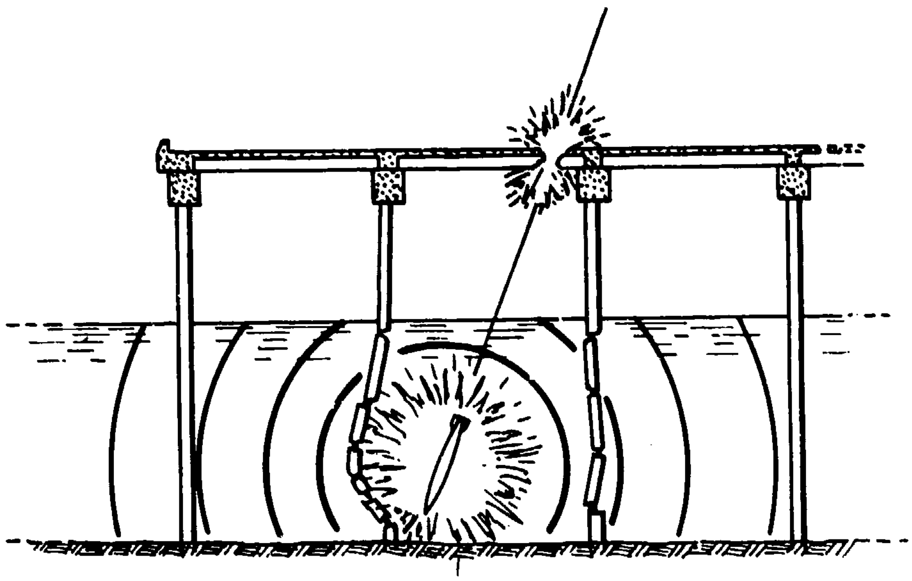


b. Deck collapse by bending failure from blast impulse load

Figure 9. Potential damage mechanisms for contact-fuzed bomb detonation against reinforced concrete slab decks of piers/wharves

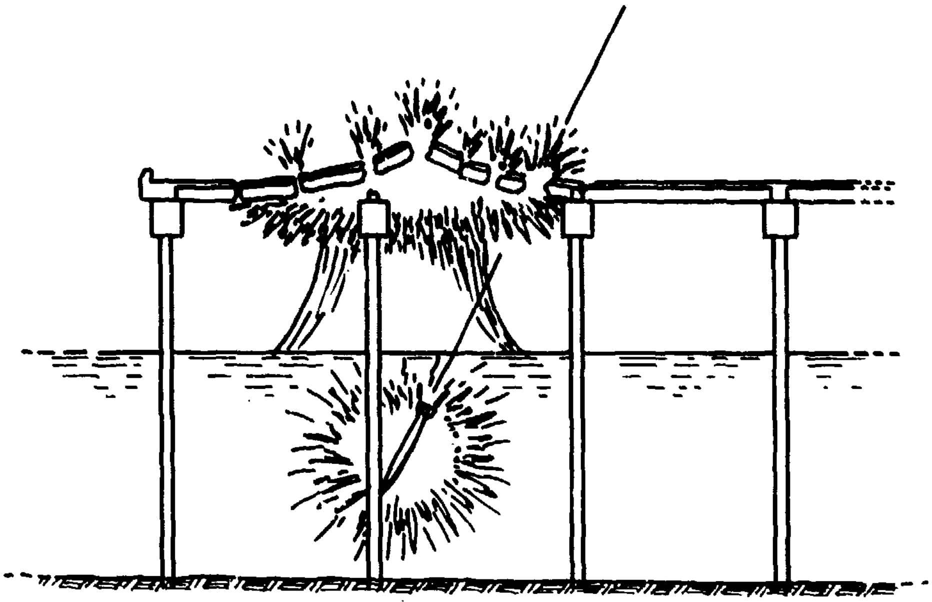


a. Air blast shock and impulse loads on pilings and deck from detonations in air below deck level

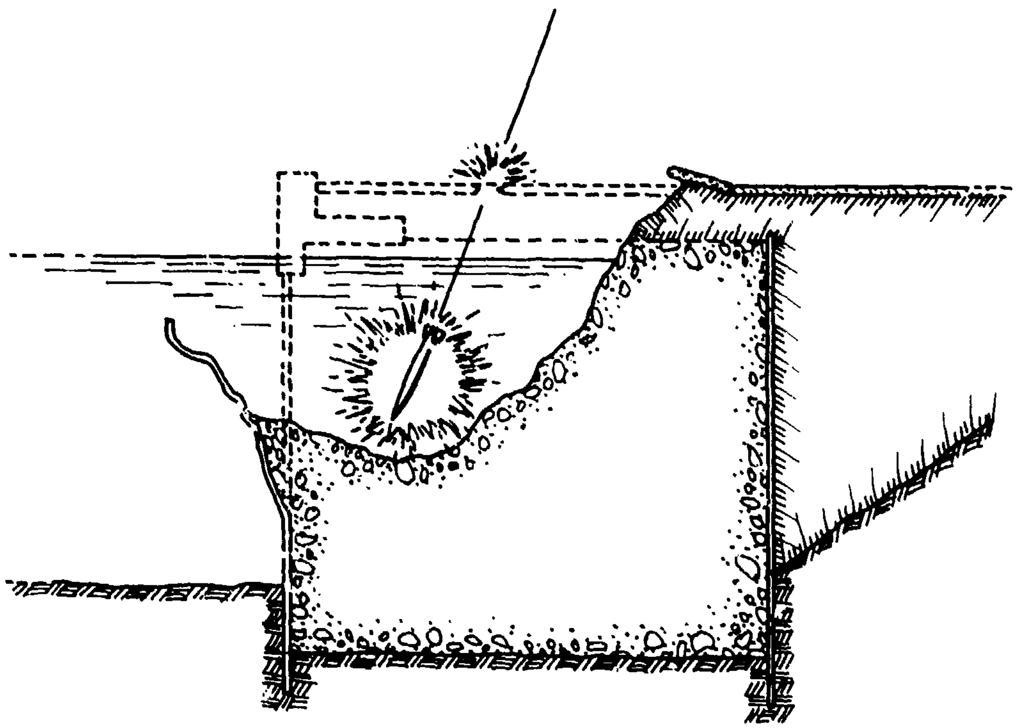


b. Water shock load against piling from underwater detonations

Figure 10. Potential damage mechanisms to piers/wharves of open construction from delay-fuzed bombs



a. Open construction, deck damage from water plume



b. Closed construction, cratering of rubble- or sand-filled steel cofferdams or sheet piling walls

Figure 11. Additional damage mechanisms from delay-fuzed bomb detonation against piers/wharves

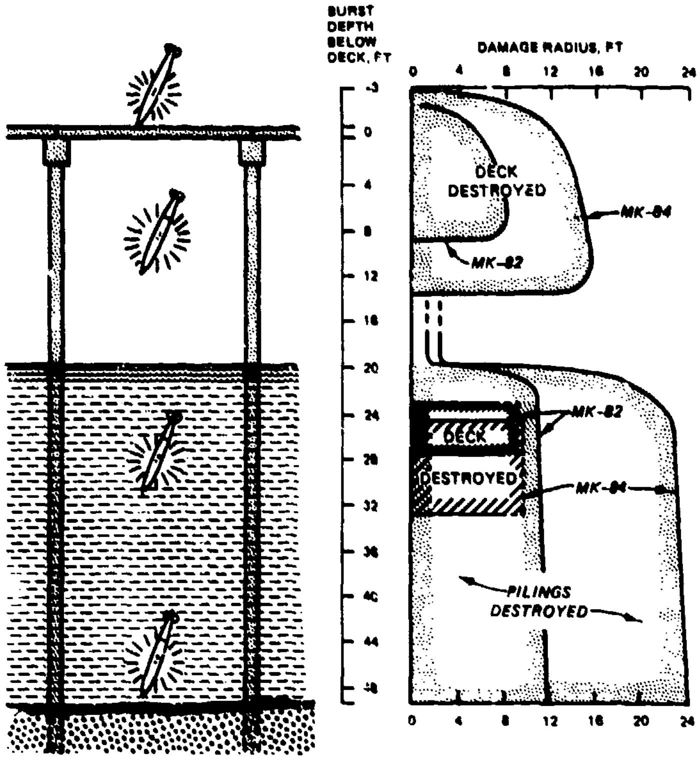


Figure 12. Damage radii for MK-82 (500-lb bomb) and MK-84 (2,000-lb bomb) warheads as a function of bomb detonation position against a typical pier/wharf with concrete or timber deck and concrete or timber pilings (MK-82 is equivalent to bomb used in analysis)

asphaltic pavement structure placed over a prepared subgrade: 12-in. subbase course, 8-in. graded aggregate base course, 0.4 gal/sq yd of R.C.-70 (prime coat), 2-1/2-in. bituminous concrete binder course, and 1-1/2-in. bituminous concrete surface course.

25. Weapon effects on asphalt paved storage areas indicate a conservative value of 44-ft true diameter for craters produced by 500-lb general purpose bombs. This value is based on an optimum depth of burst in a wet clay or sandy subsoil (Figure 16). A true crater is the hole actually excavated by the explosion but normally is covered up by all the soil and debris that fall back into the hole. The apparent crater is the visible hole left after the explosion. The dimensions for the true and apparent craters are shown in Figure 17. The true crater depth and volume are 16.5 ft and 12,544 cu ft, respectively. The apparent crater depth and volume are 9.1 ft and

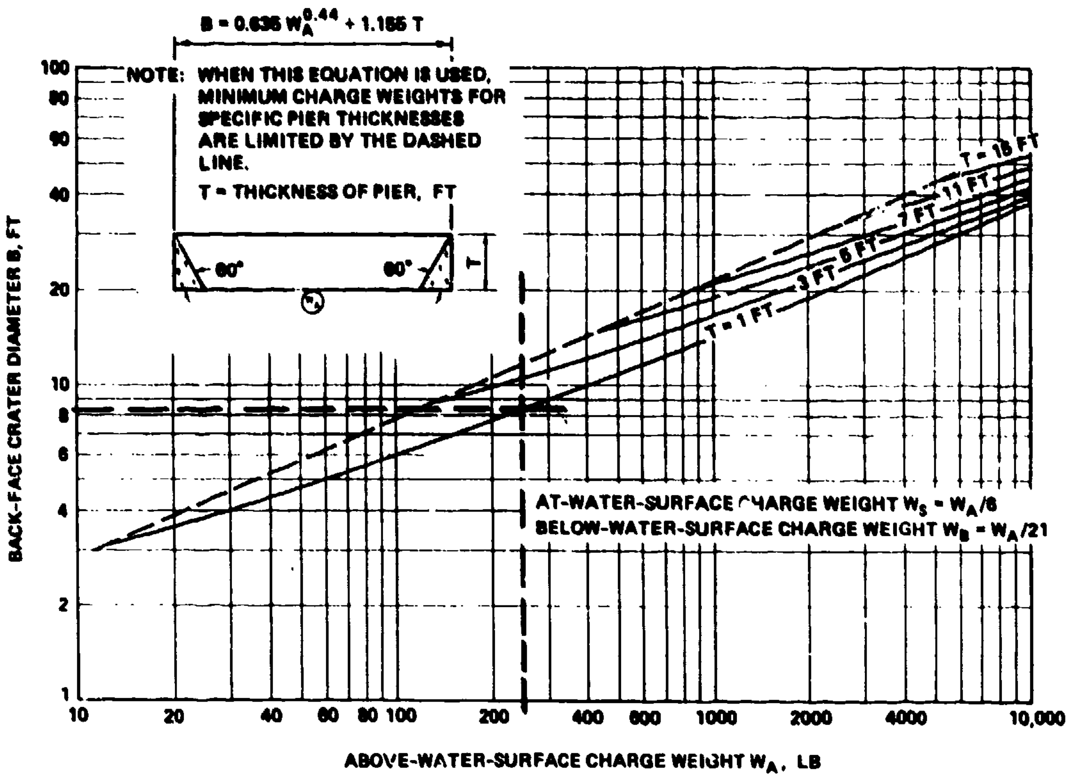


Figure 13. Diameter of back craters on concrete piers and wharves

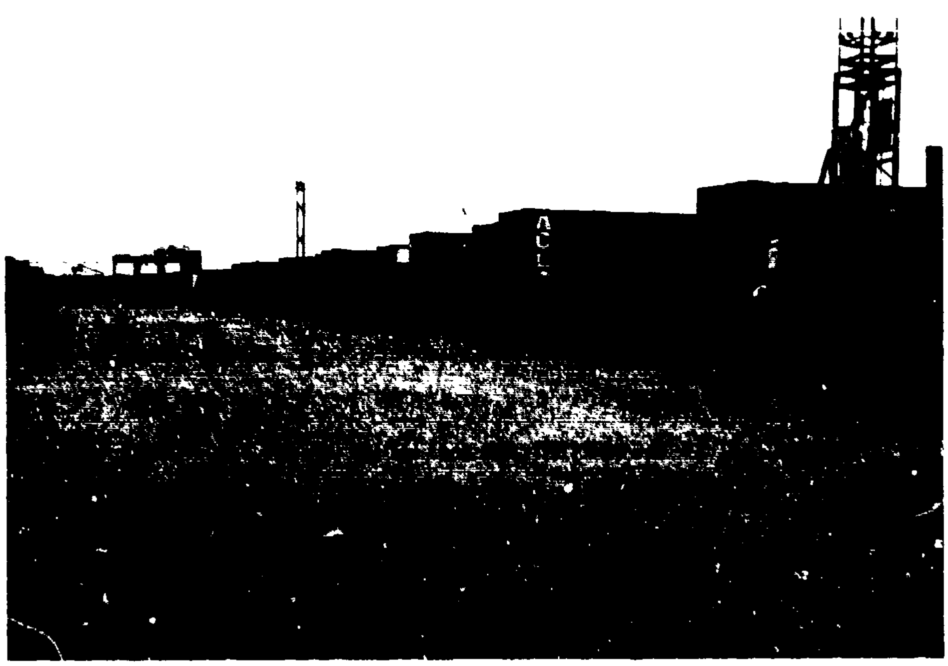


Figure 14. Containers parked in storage yards at NIT

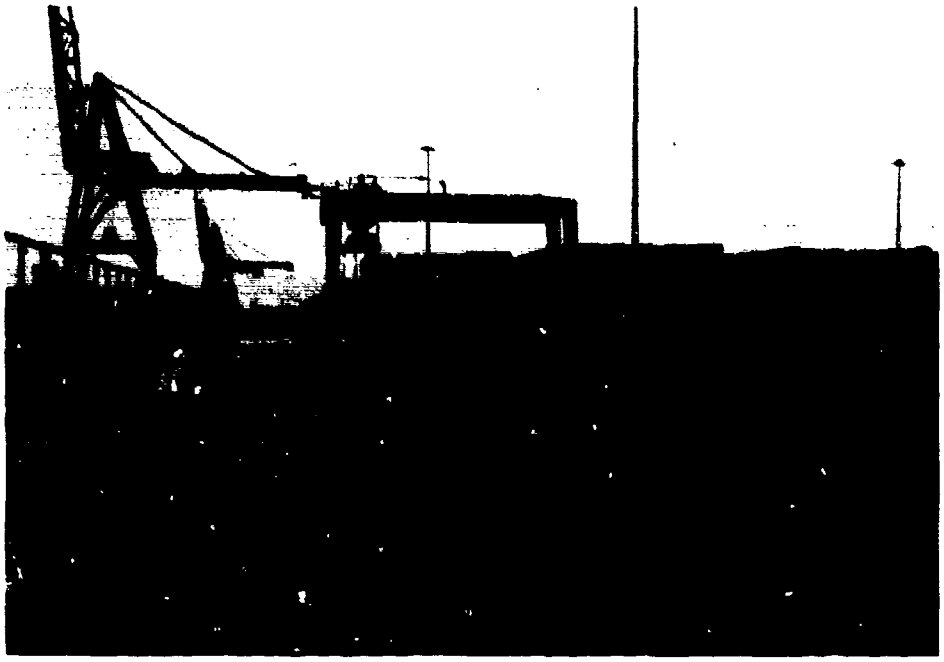


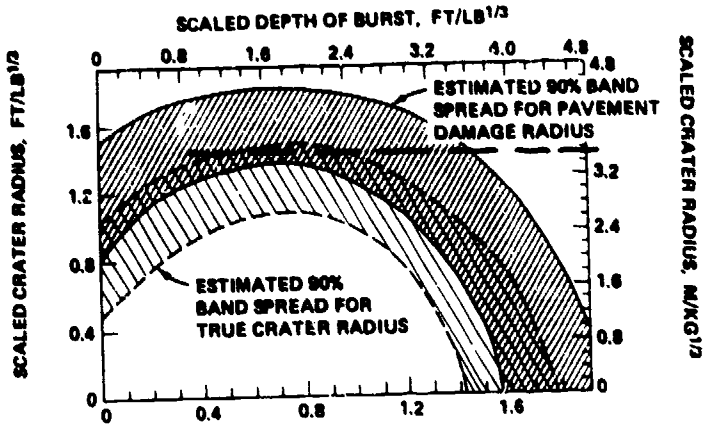
Figure 15. Containters stacked on paved surface at NIT

8,365 cu ft, respectively. These crater volumes are based upon a paraboloidal shaped segment.

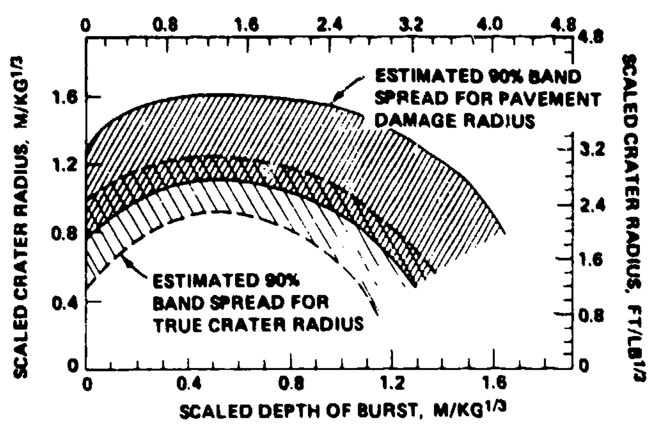
Commercial Port Target Analysis

26. Modern commercial port facilities are normally designed to handle containerized cargo with a minimum of facilities for roll-on/roll-off and bulk cargo. All of these ports have several characteristics that are almost identical in nature:

- a. Large wharf-side cranes on rails are electrically powered to handle and move containers to and from the ship. Container ships are not normally equipped to off-load themselves.
- b. Travel cranes (transtainers) on rails or tires are self-powered to move containerized cargo to and from the wharf-side crane.
- c. Large paved (usually asphalt) areas are adjacent to the wharf for staging the containers.
- d. For traffic and cargo control, there is one road for ingress and egress.
- e. One or two piers and one roll-on/roll-off wharf exist.
- f. A railroad switching yard usually exists.



a. Wet clay or sandy subsoils



b. Dry-to-moist clay or sandy subsoils

Figure 16. Estimated true crater radius from bomb detonations in asphalt storage area over wet and dry-to-moist subsoils

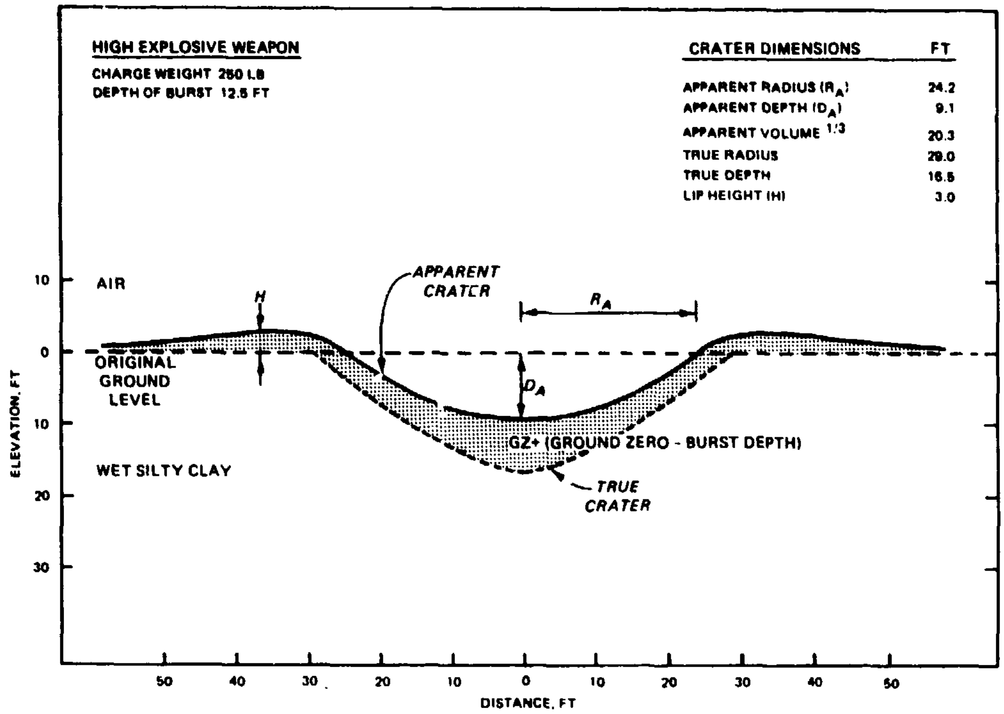


Figure 17. Dimensions of a true and apparent crater

g. Basic design is predicated on minimum time for handling cargo across the wharf.

A general view of the generic commercial port (NIT) is shown in Figure 18, and the various structural elements used as targets in this analysis are identified in Figure 19.

Attack scenario

27. An air threat with four bomber aircraft, each armed with thirty-six 500-lb general purpose bombs set for stick-bombing, was selected for the attack simulation. Eight different attack plans were initially used in the trial simulations. The headings and aim points for these attack plans were developed by four personnel to obtain a variety of ideas.

28. After analyzing the results of the first eight attack plans, conclusions indicated that the storage area was too large for a significant amount of damage to occur, and that the gate complex and entrance to the port were too small an area to be bombed efficiently. The container wharf and pier areas were chosen as primary targets because of their importance in loading and unloading cargo. Three new attack plans were drawn as a basis for this study after analyzing the results of the first eight plans.

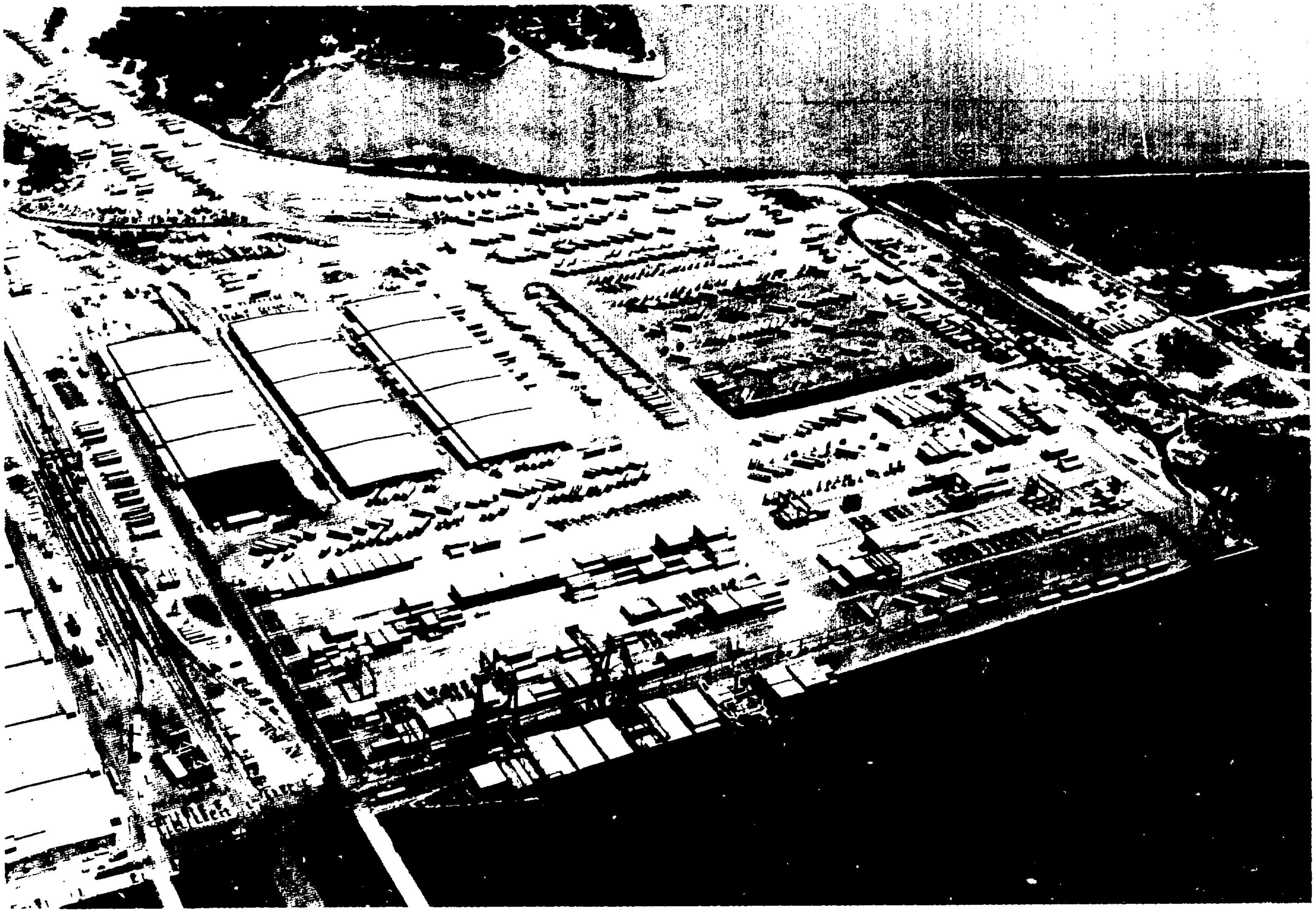
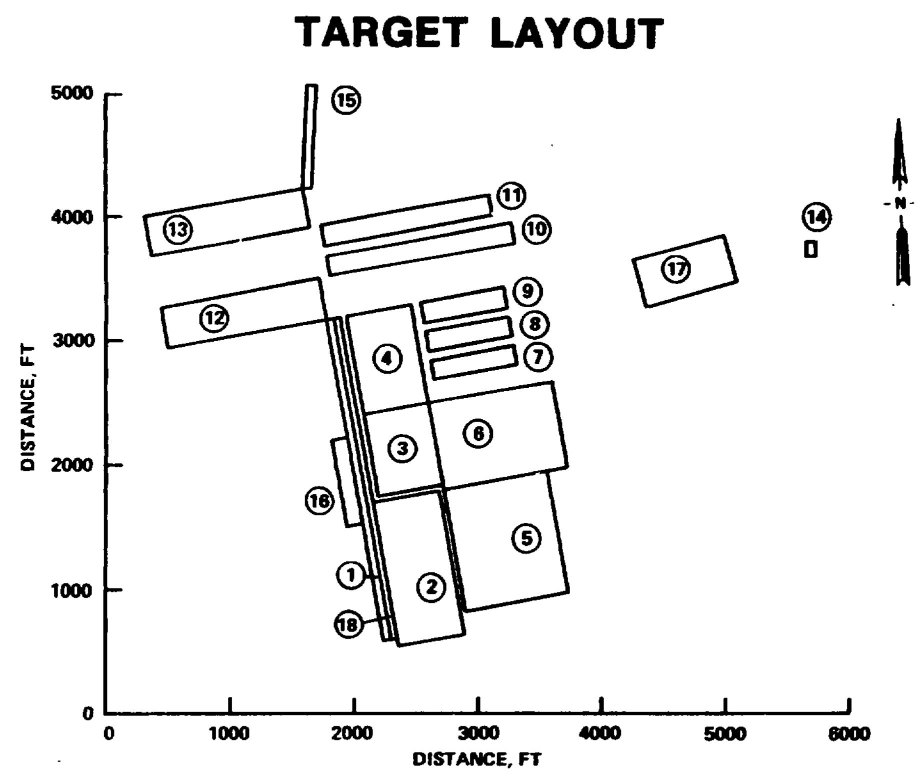


Figure 18. General view of generic port NIT (Courtesy of American Association of Port Authorities)

TARGET NO.	TARGET DESCRIPTION
	TARGET TYPE #1 – PIERS AND WHARFS
1	WHARF 1 (AREA NEXT TO THE WATER AND LOCATION OF CRANE RAILS)
18	WHARF 2 (AREA NEXT TO THE PARKING LOTS)
12	PIER 1
13	PIER 2
	TARGET TYPE #2 – STORAGE AREAS
2	TRANSTAINER STORAGE 1
3	TRANSTAINER STORAGE 2
4	TRANSTAINER STORAGE 3
5	CONTAINER–ON–CHASSIS STORAGE 1
6	CONTAINER–ON–CHASSIS STORAGE 2
	TARGET TYPE #3 – WAREHOUSES
7	WAREHOUSE 1
8	WAREHOUSE 2
9	WAREHOUSE 3
10	WAREHOUSE 4
11	WAREHOUSE 5
	TARGET TYPE #4 – ROADS
14	GATE COMPLEX
17	PLACE WHERE ROADS NECK DOWN
	TARGET TYPE #5
16	SHIP
	TARGET TYPE #6
15	NORTH BERTH
	TARGET TYPE #7
19	ENTIRE TARGET AREA



NOTE: CIRCLED NUMBERS ARE TARGET NUMBERS.

Figure 19. Generic port targets and layout

Bombing simulation results

29. Monte Carlo bombing simulations were run for each attack plan with hit and damage statistics computed. The initial eight attack plans are shown in Figures 20 and 21. The aim points and headings are shown by the arrows. The three new attack plans that were used in this analysis are shown in Figure 22, and the results of the bombing simulation are shown in Figure 23. The wharf area, targets 1 and 18, received an average of 20.8 hits. The average number of hits on the piers, targets 12 and 13, was 6.6 with a spread of 2.6 to 8.9. The asphalt pavement storage areas, target type number 2, received an average of 41.1 hits and the warehouses, target type number 3, acquired an average of 8.1 hits. Target type number 4, the roads, did not take any hits;

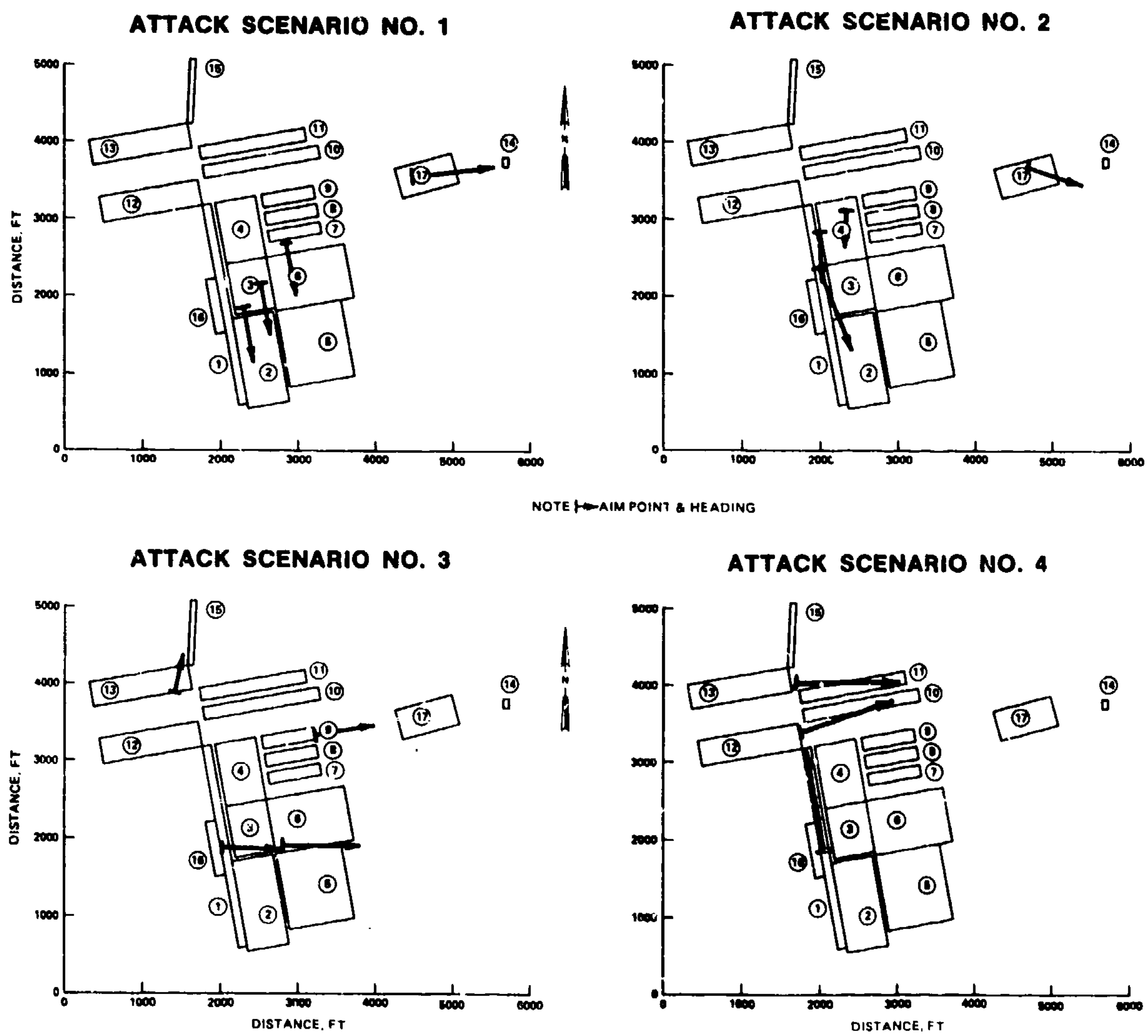


Figure 20. Initial attack scenarios 1 to 4

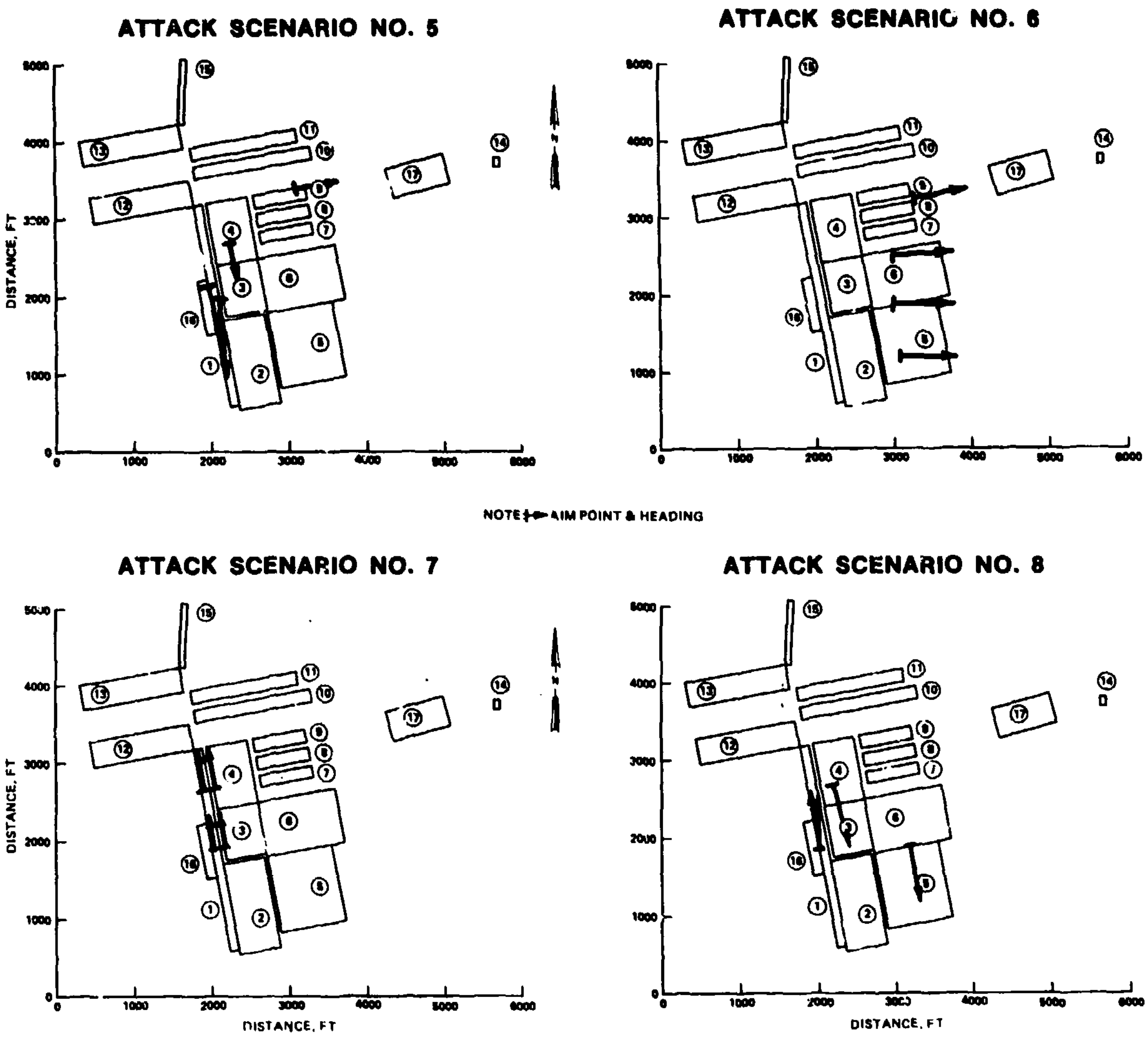


Figure 21. Initial attack scenarios 5 to 8

target type number 5, the ship, received 2.0 hits; and target type number 6, the north berth, acquired 0.8 hits.

30. Simulations were conducted with a minimum wharf length of 600 ft, either undamaged or repaired, as the criteria to service a ship. The number of holes to repair in order to service one, two, or three ships is 2, 5, or 9, respectively. Holes in the wharf decking could present cargo handling problems. Analyses indicate that 25 percent of the hits on the wharf are in the crane rail area. From this, it can be deduced that the crane and crane rails are prime targets for limiting container traffic.

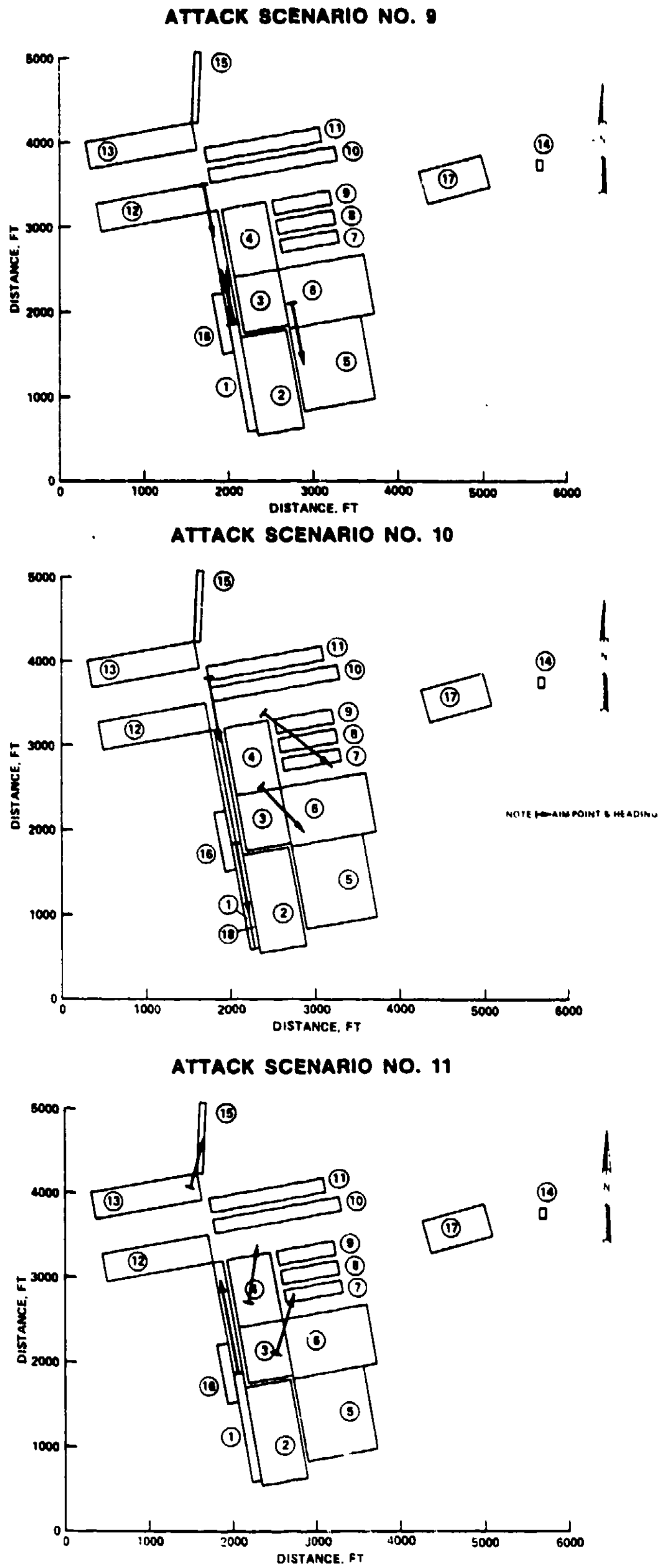


Figure 22. Attack scenarios 9 to 11
used in target analysis

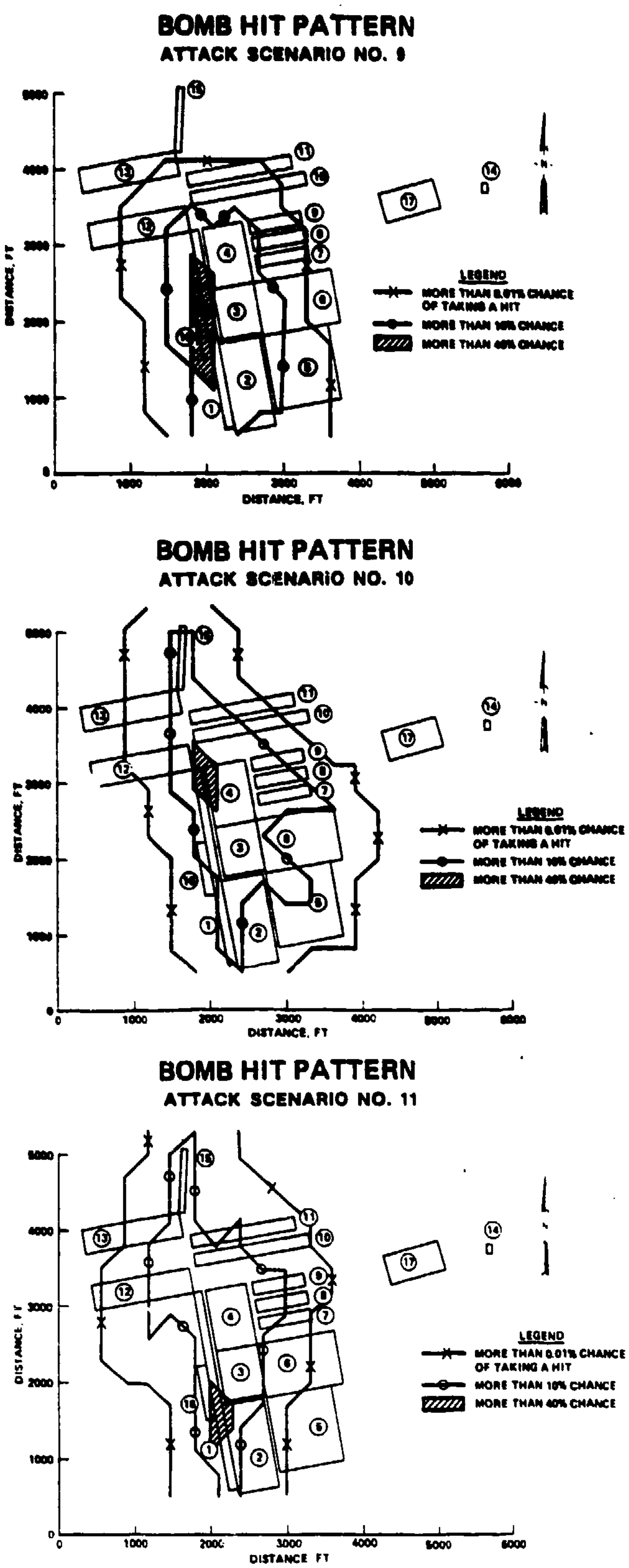


Figure 23. Simulation results from attack scenarios 9 to 11

31. Bomb craters in the storage areas are numerous, and large quantities of debris will need to be removed. Approximately 40 craters, each with a 44-ft diam, will have to be repaired.

Results from scenario changes

32. Further analyses were conducted by increasing the number of planes which led to an increase in the number of hits for most of the targets. As the number of planes was increased, the area covered was increased. These results are shown in Figure 24a. Improving the bomb delivery accuracy did not always increase the number of hits. Some targets were hit by stray bombs. As the accuracy was improved and there were less stray bombs, these targets were hit less. Therefore, some targets were hit more when the accuracy was improved, and some were hit less. The results are shown in Figure 24b. The accuracy was based upon a range error probable (REP) and a deflection error probable (DEP).

Military Port Target Analysis

33. Military port facilities are used mainly to refurbish ships, to provide ship docking for resting ship's crew, and to restock ship supplies. General cargo and limited quantities of containerized cargo are handled by military ports in areas where commercial port facilities are prevalent. All of these military ports have several characteristics that are almost identical in nature.

a. Numerous piers, some with warehouse buildings and some without.

b. Very little space between the edge of the wharf beside the water and the buildings which parallel the water's edge.

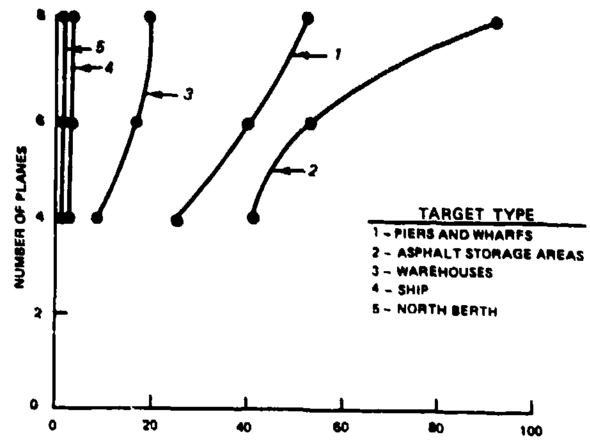
c. Wide variety of buildings which are very closely spaced.

d. For security, one main road for ingress and egress.

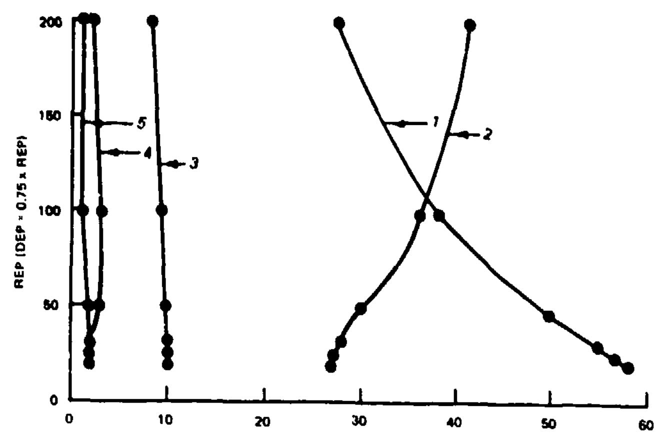
The generic military port (Norfolk Naval Base) layout and the various targeted structures used in this analysis are identified in Figure 25.

Attack scenario

34. An identical air threat as used against the commercial port and described in paragraph 27 was selected for attack simulations against the military port. Eight different attack plans were initially used in the trial bombing simulations. After analyzing the results of the first eight plans, it was decided to focus attention on the piers and the areas within 1,000 ft of



a. Increasing the number of planes

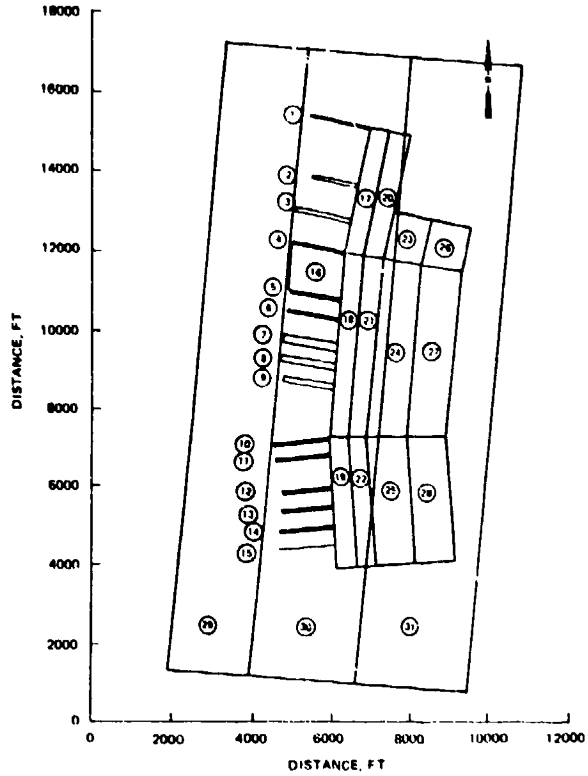


b. Improving bombing accuracy

Figure 24. Results from changing attack scenarios

TARGET NO	TARGET DESCRIPTION
	TARGET TYPE # 1 - PIERS
2	PIER 12
3	PIER 11
4	PIER 10
5	PIER 7
6	PIER 5
	TARGET TYPE # 2 - PIERS, BREAKWATER, AND SMALL CRAFT AREA
1	BREAKWATER
7	PIER 4
8	PIER 3
9	PIER 2
10	PIER 25
11	PIER 24
12	PIER 23
13	PIER 22
14	PIER 21
15	PIER 20
16	SMALL CRAFT AREA
	TARGET TYPE # 3 - 1000 FEET CLOSEST TO WATER'S EDGE
17	FIRST 500 FEET - TOP
18	FIRST 500 FEET - MIDDLE
19	FIRST 500 FEET - BOTTOM
20	SECOND 500 FEET - TOP
21	SECOND 500 FEET - MIDDLE
22	SECOND 500 FEET - BOTTOM
	TARGET TYPE # 4 - SECOND 1000 FEET FROM WATER'S EDGE
23	TOP
24	MIDDLE
25	BOTTOM
	TARGET TYPE # 5 - THIRD 1000 FEET FROM WATER'S EDGE
26	TOP
27	MIDDLE
28	BOTTOM
	TARGET TYPE # 6 - ENTIRE TARGET AREA (USED TO COUNT BOMBS)
29	AREA 1
30	AREA 2
31	AREA 3

TARGET LAYOUT



NOTE: CIRCLED NUMBERS ARE TARGET NUMBERS

Figure 25. Generic military port layout and targets

the water as prime targets. These target areas would disrupt port operations more than other targeted areas. Three new attack plans were drawn as a basis for this study.

Bombing simulation results

35. Monte Carlo bombing simulations were run for each attack plan with hit and damage statistics computed. The initial eight attack plans are shown in Figures 26 and 27. The three new attack plans that were used in this analysis are shown in Figure 28. The results of the bombing simulation are shown in Figure 29.

36. The military port was so large that it was difficult to bomb the port efficiently with the given number of aircraft. The piers were also difficult to hit. The average number of hits on each pier was 0.44. The number of hits on each pier ranged from 0 to 1.39. The small craft area had 3.71 hits and the breakwater had 0.03 hits. There were 72 hits on the first 500 ft of area alongside the water and 29 hits on the next 500 ft. The next

Figure 26. Initial attack scenarios 1 to 4

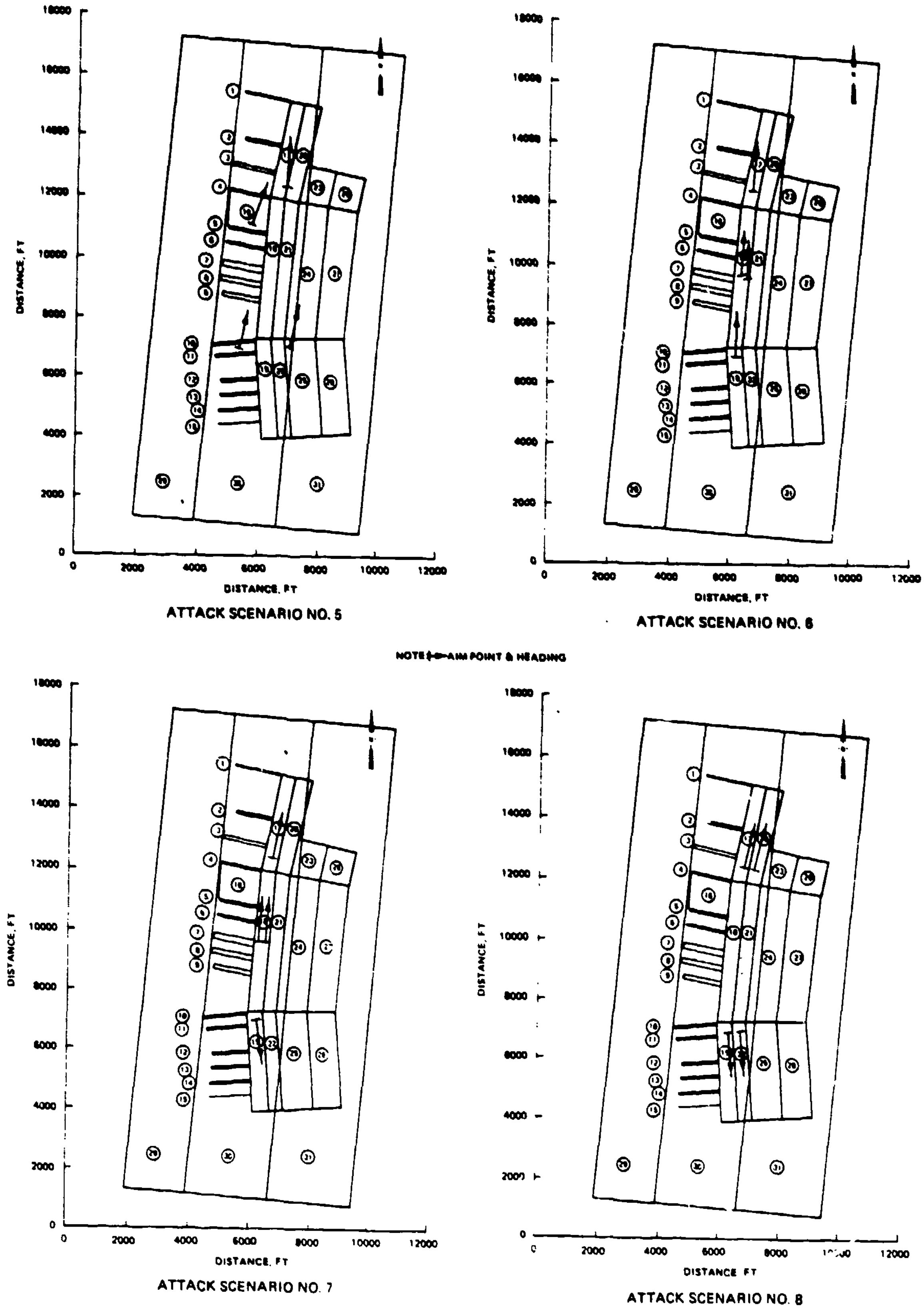


Figure 27. Initial attack scenarios 5 to 8

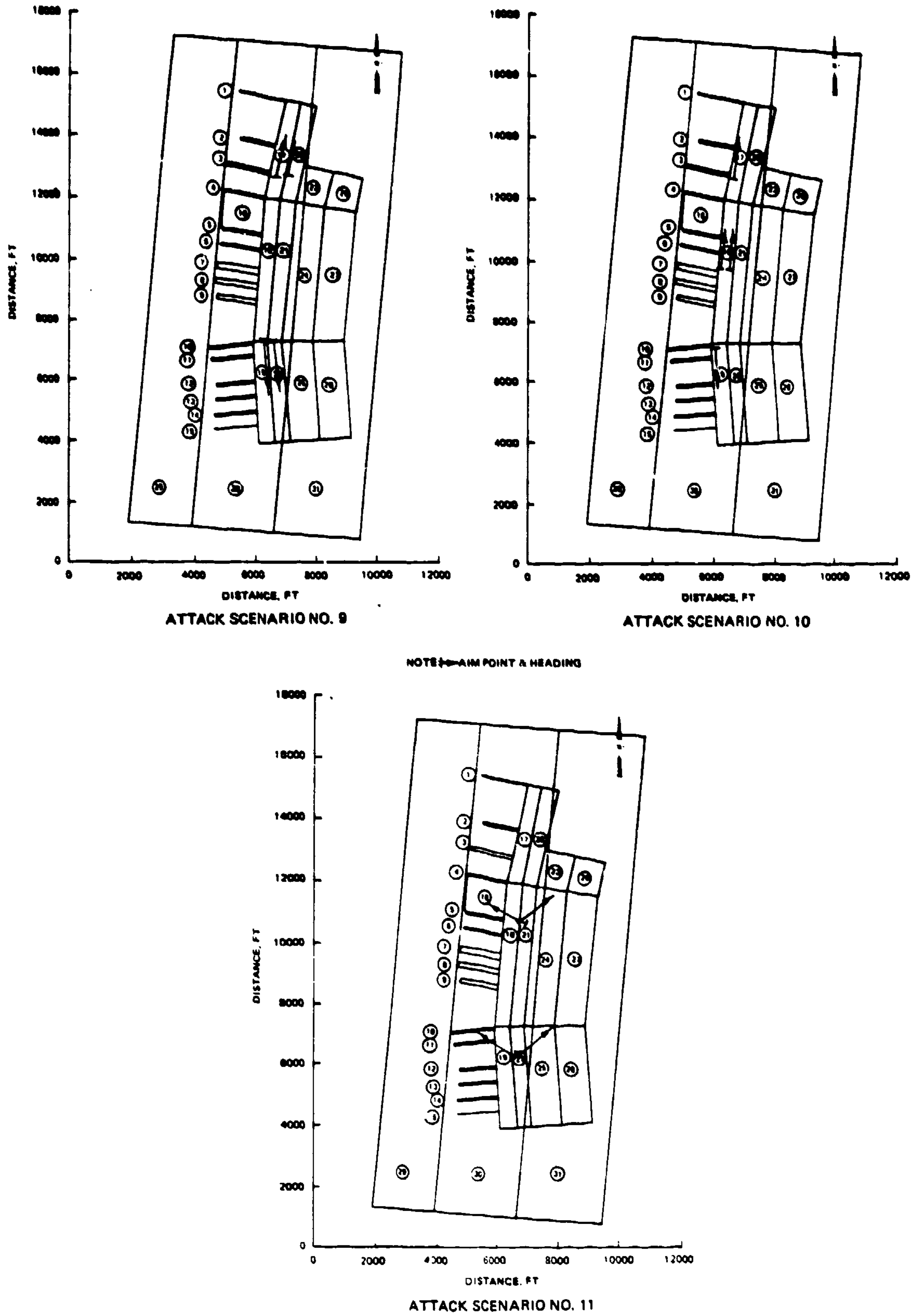


Figure 28. Attack scenarios 9 to 11 used in target analysis

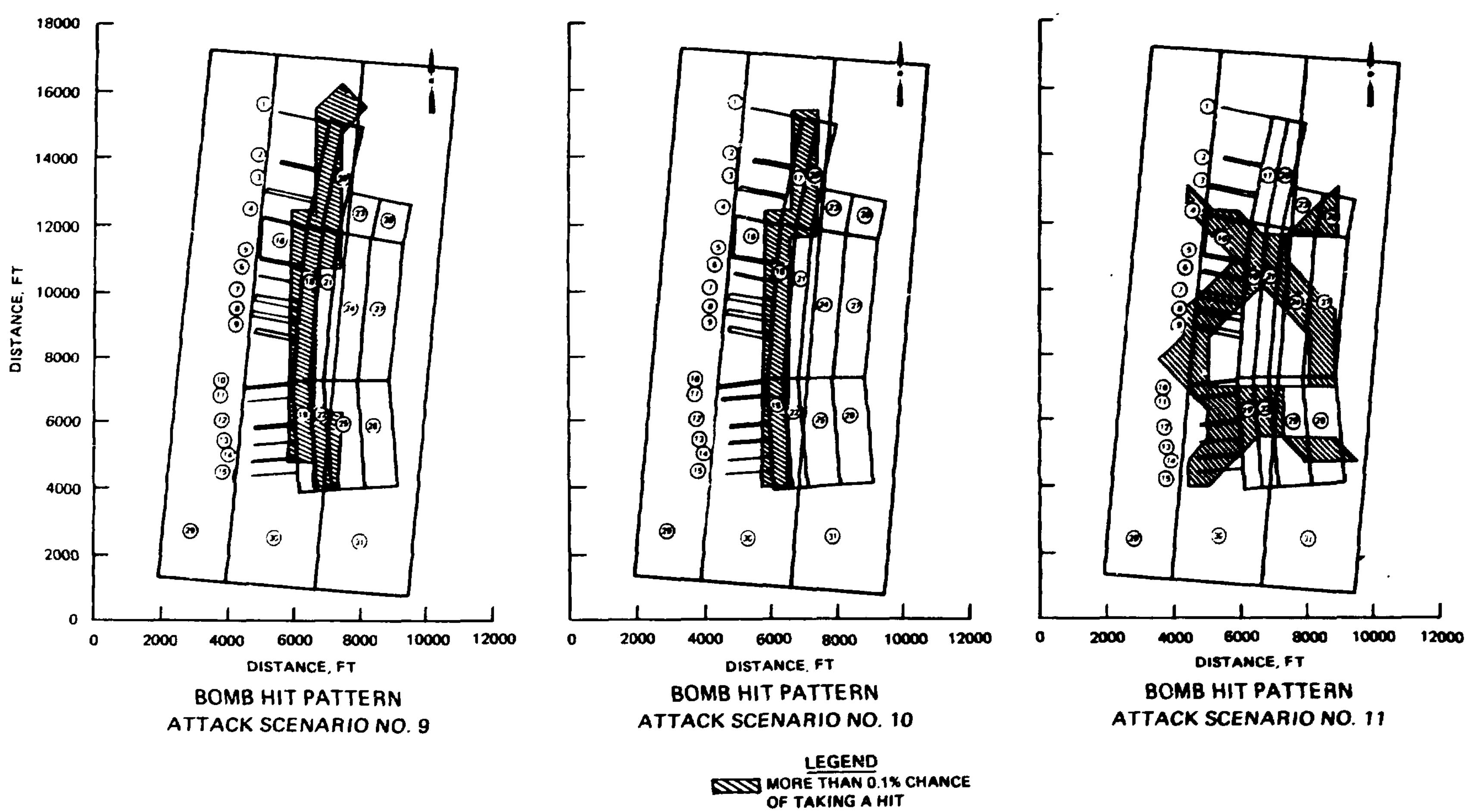


Figure 29. Simulation results from attack scenarios 9 to 11

1,000 ft only had 0.4 hits, and the next 1,000 ft did not take any hits. A total of 31 bombs landed in the water surrounding the piers would cause some problems. These bombs could damage ships in the port which would make access to the port difficult. The bombs could also explode in the water under the piers which could generate deck damage from water plume or pile damage from water shock load. Since the port is so crowded with buildings, rubble from the large number of hits within the first 500 ft of the water's edge will limit access to the piers that were not hit.

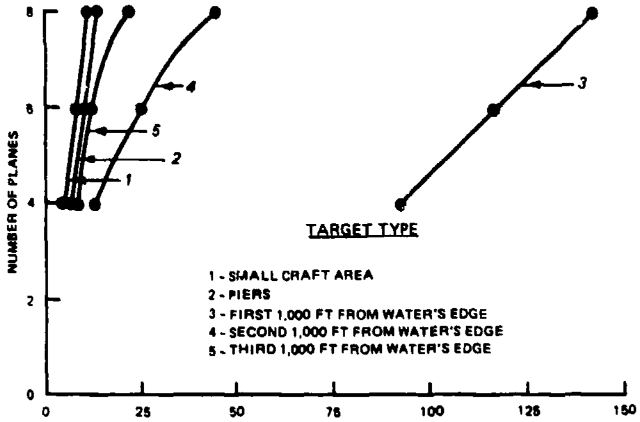
Results from scenario changes

37. Analyses were conducted by increasing the number of planes which led to an increase in the number of hits for most of the targets. The results are shown in Figure 30a. Improving the accuracy of the bomb delivery did not change the number of hits significantly. Due to the large target area, bomb accuracy changes do not change the number of hits. The results are shown in Figure 30b.

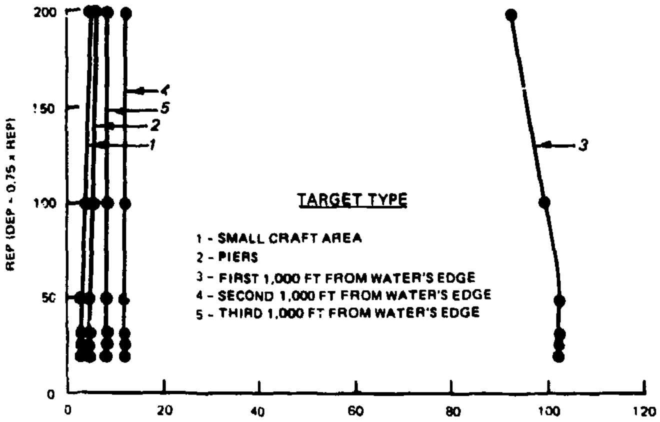
Port Vulnerability Conclusions

38. It appears that pier/wharf areas closest to the water should be of major concern and prime targets for military attacks. Disruptions of these cargo transfer facilities would hinder the operations of the port more than any other areas within a port complex. The pier/wharf areas would also be more difficult to repair. Areas adjacent to the water's edge would present rubble removal problems and also crater repair. Although this study addressed only structural damage, the attack simulations should also indicate that resultant fire and the disruption of essential utilities could be as serious as the structural damage experienced in an attack. The results of this study were used as criteria for the expedient repair of war-damaged piers/wharves, storage areas, and hardstands.

39. Increasing the number of planes lead to an increased number of hits in most cases. As the number of planes were increased, the overall area covered in an attack was increased. However, improving the accuracy of bombing did not always increase the number of hits on a given target. If the target area was large compared to the delivery accuracy, improving the accuracy did not have much effect. If the target area was small compared to the delivery accuracy, sometimes the number of hits on major targets increased, and



a. Increasing the number of planes



b. Improving bombing accuracy

Figure 30. Results from changing attack scenarios

sometimes it decreased. The surrounding targets were hit by stray bombs when the delivery accuracy was not very good. Smaller targets compared to the delivery accuracy were most difficult to hit.

PART III: PIER/WHARF REPAIR

Introduction

Purpose

40. The purpose of this part of the report is to outline the procedures, materials, techniques, and manpower necessary to expediently repair piers/wharves that have been damaged by enemy forces. The discussion within this part deals with the methods or repairs that are suitable for use by trained personnel in the theater of operations. There are many techniques and materials that are applicable to repairing damaged port structures. Since their complexity and within-theater material scarcity limit their practicality for use by the military in port repair operations, these repair methods are not discussed.

Responsibility

41. The repair techniques for a theater situation will be influenced by the capabilities and equipment structure of the troops which are responsible for repairing the port area. The engineer unit selected to repair a port depends upon the magnitude of the damage to the port. Minor damage will normally be repaired by a port construction company. Major damage will be repaired by an engineer group, an organization individually tailored for a specific mission. Host nation support may also be available. For purposes of accurately identifying the capabilities of these units, their force and equipment structure should be used as the definition of their capability, and then any additional requirements should be identified and described. These capabilities may be obtained from additional support units or personnel.

Scope

42. Pier/wharf construction generally consists of decking supported by pile foundation structures. The equipment which is found on piers/wharves can range from warehouses for storing bulk goods out of the weather to large gantry cranes used to unload cargo ships. Although pile supported structures are the general case, some type of pier/wharf structures are either founded on backfill material which has been placed out into deep water or rockfill material used to support the decking.

Repair decisions

43. The condition of a structure which has undergone some amount of damage can be placed into three categories: (a) portions of the structure must be replaced because of their structural load-carrying capacity being destroyed, (b) portions of the structure have only received cosmetic damage and do not need any structural repair, and (c) those portions of the structure which are damaged but do not need to be entirely replaced can be repaired. Those portions that must be replaced should be replaced if replacement materials and equipment are available. However, if a structure can be repaired, a decision must be made whether to repair or replace it. It is not always apparent which is the most expedient solution. If replacement is more expedient, or replacement materials more available, then it may be more advantageous in terms of time taken to replace rather than repair. These decisions will need to be made at the time of structural assessment.

Port construction company capabilities

44. At present the 497th Engineer Company is the active Army port construction company stationed at Fort Eustis, VA. Two port construction Army Reserve companies also exist: one located in California and the other in Puerto Rico. At its highest strength level, the active Army company has attached 214 personnel, and adequate equipment and expertise to conduct over-water pile driving, install timber dolphins, construct floating offshore petroleum, oil, and lubricant (POL) pipelines, erect the Army's DeLong piers, place mooring points (up to 1 mile offshore), be responsible for underwater construction and surveys, conduct powered vessel operations, conduct limited dredging, erect the Navy Elevated Causeway System (ELCAS), and provide manpower for land and over-water conventional construction. Limitations of the company include: insufficient training time in heavy marine construction, limited in training scenarios which simulate container port repair, insufficient training in concrete construction, and rotation of personnel which inhibit marine construction expertise.

Port construction company equipment

45. The company has adequate vehicles to haul equipment and supplies to the construction site as well as remove rubble and debris that will not be reused. The company has a diving section that is equipped for underwater construction and demolition. The general construction section has equipment and personnel to accomplish most above-water repair and construction tasks. The

company is limited to old equipment in need of repair. The table of organization and equipment (TOE) for the port construction company is described in TOE 5-129H (Headquarters, Department of the Army 1975b).

Support from other units

46. The engineer group could typically consist of a port construction company, pipeline construction and support company, combat heavy engineer battalion, dump truck company, engineer construction support company, dredge teams, and other units as required. Within the combat heavy engineer battalion are the following capabilities relevant to port construction activities: rock production/quarry section with a capacity to process 75 tons of aggregate per hour and direct support unit (DSU) maintenance shop with welding capability, the Engineer Construction Support Company has the capability to crush rock for producing concrete aggregate, the Engineer Dump Truck Company can move bulk materials in quantities up to 312 cu yd in one move, and the Engineer Pipeline Construction Support Company has the capability to construct pipelines up to 24 in. in diameter for carrying water or petroleum products from offshore to the field. There are a number of engineer teams which can be attached to the engineer group to augment the capabilities to conduct port repair. These include a diving team of nine personnel for underwater rehabilitation, construction, or demolition; a team of qualified welders with equipment beyond the organic assignment; and dredging teams of from 42 to 98 personnel for dredging operations in the port environment. Other units that can assist in port repair include the Engineer Panel Bridge Company and the Engineer Float Bridge Company.

Type B units

47. The organization of the Port Construction Company under a Type B strength consists of the minimum number of military personnel for command and control of the unit augmented with non US personnel or third country personnel to conduct the work of the company. The military strength of the Port Construction Company under Type B organization is 157 personnel. Extensive use of civilian personnel (both US and indigenous) is recommended during time of war to allow key military personnel to accomplish other more important tasks and to utilize the specialized skills of people trained in port construction tasks to their fullest advantage.

Statement of the Problem

48. The amount and type of damage that can be expected at a port site will vary considerably depending on the circumstances through which the port was acquired and the intensity of the conflict causing the damage. The amount of repair and the types of repair methods will depend, to a great extent, on the damage occurred. The details of this threat were discussed in Part II for the entire port area. For purposes of the discussion that follow, only the pertinent facts related to piers/wharves are reviewed. Anticipated container port damage due to military action is depicted in an artist's concept (Figure 31).

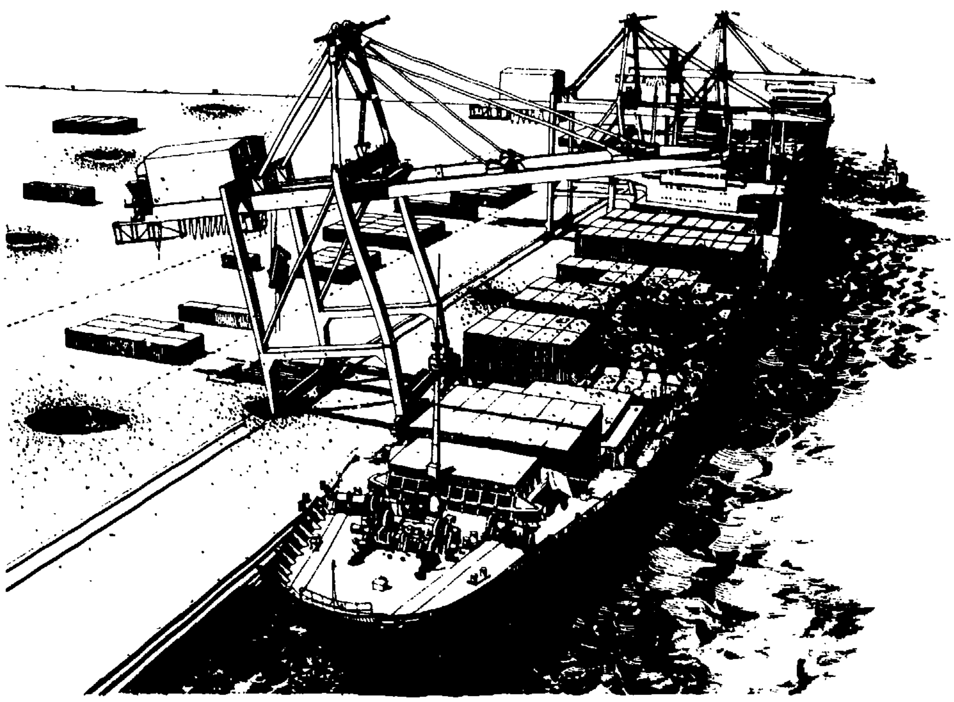


Figure 31. Artist's concept of a war-damaged container port

49. The heaviest damage to a port area will probably occur on wharves. It has been determined that wharves which run parallel to the shoreline sustain more damage than piers that extend out into the water either perpendicular to the shore or at some angle to the shore.

50. The primary emphasis of aerial bombing is probably to destroy ships docked at piers/wharves. These bombing runs are not entirely accurate, and a certain percentage of the bombs meant for the ships will strike the piers/ wharves adjacent to the ships. Additionally, the wharves will be targets

themselves. They will have cargo handling equipment positioned on them which, if destroyed, will halt wharf operation. As a result of these considerations, there will likely be a number of locations along the wharf that become damaged as a result of aerial bombing.

51. The damage to any port structure as a result of bombing is due to blast effect and the damage resulting from fire. The results of blast damage will be a function of the type and size of bomb; however, the damage caused by fire is a function of the combustible materials in the vicinity of the blast area. The resulting fire will cause damage to all types of structures (whether wood, concrete, or steel). The damage to wood structures will be obvious, and that to steel will be easy to recognize, but the damage to concrete will not necessarily be obvious unless the intensity of the fire is severe.

52. The bombs which will be used in a conventional attack will normally be activated by surface contact and will detonate immediately. This will cause damage to all horizontal surfaces within the impact area, thereby limiting damage to supporting structure (i.e., beams, columns, and piles) depending on the proximity of the structure to the center of the blast.

53. The worst case of damage from one bomb would be a direct hit on one pile cap of a wharf. The effect of a direct hit on a pile cap would be to destroy that pile cap and the supporting structure beneath it. If the bomb hits directly on the cap, it would have a destructive effect on all four slab elements (or bays) which are framed into that pile cap. Since the supporting structure has been destroyed, the slabs would break due to bending loads which are imposed as a result of the loss of support from the framing members. This scenario would effectively destroy all four bays which are framed into the pile cap.

Resources for port repair

54. Troops. The number of units which will be assigned to the task of repairing a port area will depend upon the urgency and the amount of damage incurred. One port construction company will not have adequate personnel and equipment to repair massive damage. These resources will need to be augmented by additional troops if the task is large and the time frame constrictive. The use of additional troops from other construction companies such as combat heavy or light units will provide personnel who are skilled in construction techniques and will be familiar with the equipment assigned to the port

construction company. An additional source will be host nation personnel. They can be hired to provide labor and some small degree of skilled labor to allow those trained troops to conduct more skilled tasks. The demand for local personnel will be great and their allocation will greatly depend on priority.

55. Equipment. The equipment inventory in the TOE for a port construction company is heavily oriented towards repair and construction of above-water wooden structures. This is apparent by the number of trucks, cranes, bulldozers and general construction related tools and equipment in the TOE. The personnel and equipment dedicated to water related work consists of divers, power boat personnel, pipeline construction equipment, and pile driving equipment. Many ports around the world are constructed using wood as the major construction material; however, there are a number of piers/wharves that are constructed of concrete or composite concrete and wood materials. The concrete related repair and construction equipment integral to a port construction company is limited to a 2-cu yd concrete bucket, one 8-cu yd mobile concrete mixer truck, 2 pneumatic concrete vibrators, and 2 concrete and masonry tool kits. Using the 8-cu yd mobile mixer to provide concrete for any placement, it is unlikely that more than 2 cu yd of concrete could be placed in a 1-hr period. At this rate it would take approximately 8 hr to place a 20-ft by 20-ft by 12-in. concrete slab. Considering that this is the damage that would be done by one bomb at one location in a port environment, the process of repairing many damaged areas would be exceedingly slow at the present level of concrete equipment authorization. Even with the attached personnel and equipment of a Quarrying and Rock Processing Team and contracted and indigenous civilian concrete personnel, a major rehabilitation of a port area will be a formidable task.

56. Materials. The three major construction materials for piers/wharves are wood, concrete, and steel. The availability of these materials in a theater of operations is a function of the specific location of the theater. Fortunately piers/wharves are most likely to be constructed from materials which are in abundance locally, and there is a good chance that there will be a supply of these materials available for repair. Wood is the most abundant of the above materials, and the majority of port structures are made from wood and wood products. Not only are they abundant, but also they are resistant to corrosive and weathering effects which are prevalent in the marine

environment. In urban areas especially in developed countries, concrete is generally available for construction, and it is more likely to be used in port construction work because of its durability and strength. Since concrete is a valuable construction material, it is likely that during wartime it will be in great demand but less accessible due to interruption of supply lines. For these reasons repair techniques rather than replacement techniques should be exercised wherever possible. Steel is the least used construction material in a port area. Not only is steel expensive, but also it has high maintenance requirements. Steel is susceptible to corrosion in a salt-water environment. Unless steel has been obviously damaged by blast impact or intense heat has caused it to soften and deform, it is likely that the steel is usable without being damaged. Repair techniques for steel construction mainly consist of welding, bolting, and riveting.

57. Expertise. The port construction company alone has the expertise of conducting construction and repairing operations onshore, at the water's edge, and overwater and underwater. However, since the company is equipped for overwater and underwater construction where other Army units are not, its expertise in this area should be exploited to the fullest. Since a port construction company is so small, it should be utilized in pile driving and repair work, underwater construction and debris removal, underwater pipeline construction, and any above-water construction or debris removal work which requires the use of floating cranes or other lifting equipment. Work that can be accomplished by troops on land should be done by heavy construction personnel who are trained in these fields.

Liabilities

58. Time. Time is more likely to be a liability than an asset in port rehabilitation work. The speed with which offensive operations need to be accomplished requires immediate supply lines, and the time it takes to thoroughly repair a port environment will not support these needs. Expedient efforts need to be followed in order to make the port area usable in the shortest amount of time. Port reconstruction is normally conducted in three phases (Headquarters, Department of the Army 1964). In the first phase salvage operations are conducted, and engineer construction units plan what efforts must be made to reconstruct the port. In the second phase structures are rehabilitated or expedient methods are developed for moving cargo and repairing existing structures. Repairs to existing structures are made using

techniques that are rapid and are not expected to last for more than 5 years. These techniques reflect more on repair than replacement. The repair techniques given in this section are not meant to be permanent repairs, but those that will put the structure back into operation in the shortest time span as well as perform the length of time required by the military. In the third phase more permanent construction is undertaken.

59. Materials. While there may be the required quantities of the needed materials to repair port structures, the quality of these materials may be below standard, producing an inferior product. The wood products in various areas of the world are of inferior quality or in some areas are nonexistent. In the Middle East the soil characteristics are not compatible with the growing of trees; consequently, there is a severe shortage of wood products. These commodities are normally imported. Where they are taken from local sources, the strength and durability of the woods will be inferior. As a result of the lack of wood products, much of the construction in this area is concrete. Cement, however, must be imported into the area since it is not a material that can be reliably obtained from local sources. Reinforcement steel obtained from local sources may not be of high quality as that obtained in the United States (US). The quality of aggregates which are used in concrete will depend upon the source from which they were quarried. In coastal areas the quality of local sand may be contaminated by sea salts. The grading of the sand will also be a factor in the quality of concrete made from this material. The engineer will need to be aware of these shortcomings and compensate for them in his design.

Expected Loads

60. The repair of war-damaged pier/wharf facilities requires a knowledge of the maximum loads for which the structure was originally designed. The structure should be restored to its original structural strength in order for it to function in the same capacity as it once did and to be able to utilize any dockside equipment that it once had such as railroad cars or rail-mounted cranes. These loads are not always obvious, and it is doubtful that design loading figures will be available to the repair team.

61. It is also quite possible that the loads for which the pier/wharf was designed will not be adequate to support the loads of the modern day Army

equipment for containerization. The age of the facility will have a large effect on its load-carrying capacity. Piers/wharves in small developing countries as well as older structures in the larger countries were originally designed to handle break bulk cargos which did not require the large container gantry cranes of today's container ports. Nor did they have to support the large concentrated loads associated with such a port. For these reasons it is helpful to be aware of some loadings which a pier or wharf is likely to be subjected in the course of handling the cargo necessary to supply an armed force.

Types of loads

62. There are two types of loadings which a pier has to resist. These are general area loadings or those loadings which are likely to exist at any spot on the wharf, such as forklift trucks, stacked containers, trucks, and other vehicles which pass through a port area; and specific area loadings or those that will only be expected to exist at a known specific area or areas, such as railroad and crane loadings. The general area loadings covered herein are trucks and other military vehicles and material handling equipment (such as forklift trucks and stacked materials which include containers, bulk cargo, and oversized loads). The specific area loadings are confined to crane and rail traffic loads.

Loading considerations

63. When repairing an area of a pier/wharf or upgrading it to carry heavier loads, the undamaged areas must be upgraded to the specifications of the repaired areas if heavier vehicles or loads will be placed on these undamaged areas in the course of moving goods through the port area. The minimum design dead loads are those which are calculated in accordance with the weights which will always be on the pier. These should be calculated by the engineer in charge of the project. The live loads are those which will be imposed on the structure for some short time such as the loads of a forklift truck rolling over the pier. These can be anticipated and incorporated into the design. Minimum design live loads for general cargo are from 600 to 1,000 psf; the design live loads for pipe and metals is 1,000 psf; those for the storage of ammunition are approximately 2,000 psf. In general, piers/wharves are designed to handle the loadings of large container handling equipment and are presently designed for loads of over 1,000 psf.

General area loads

64. Trucks. Trucks will not be considered dead loads since they will not be stored on the pier/wharf for extended periods of time. The Army has many vehicles in its inventory which will be off-loaded across pier structures. The weights of some of these vehicles are given in Table 1. These weights are considered general area loads since they can be found on any area of the structure. The weights given must be divided by the proportional weight given to each wheel in order to obtain point loadings on the pier. The tractor trucks mentioned in this table are the ones which will normally be called upon to transport military containers loaded on trailers from the off-loading site at the pier to the storage yards behind the port.

Table 1

Weights of Military Truck Vehicles

Type	Weight, lb
3/4-ton cargo truck	7,400
1-1/4-ton cargo truck	9,600
2-1/2-ton cargo truck	17,000-19,000
5-ton cargo truck	25,000-30,000
8-ton cargo truck	43,000
10-ton cargo truck	50,000
M48, M221, M275 2-1/2-ton tractor truck	18,500-19,500
M52, M52A1 5-ton tractor truck	34,500
M123 10-ton tractor truck	62,200
M26A1 12-ton tractor truck	48,900
M915 linehaul tractor truck	18,600
M878 yard tractor truck	16,300
16-ton, 6 × 6, CONEX transporter	76,000
M871 trailer w/20 ft container	57,600
M872 trailer w/2-20 ft container	106,600

65. Other critical vehicles on the dock. Aside from transfer vehicles being present on the piers/wharves, there will be heavier vehicles that must be throughput into the port area. Some of the heavier vehicles will be

construction equipment. The weights of some of these vehicles are listed in Table 2. These vehicles do not need to have access to every portion of the pier area unless they are being used to repair the pier. In most cases they can be restricted to areas which have been reinforced to handle their weight. Tanks, being critical weapons, would most likely be air lifted to the theater; however, their weights are given for convenience from approximately 60 to 105 ton.

Table 2

Weights of Construction and Bridging Vehicles

Type	Weight, kips
Construction Vehicles	
M6, M8 Bulldozer	107.8
M9 Bulldozer (tank mounted)	116.0
Earthmoving tractor	54.2
Full tracked tractor	16.0-48.0
2-1/2-ton dump truck	20.0
5-ton dump truck	32.7
40-ton shovel crane	132.6
40-ton crawler mounted, revolving	103.0
5-ton wheel mounted crane	16.0
20-ton wheel mounted crane	60.0
Bucket loading crane	23.0
Bridging Vehicles	
AVLB (M48) chassis	96.0
AVLB (M48) w/60 ft bridge	128.0
AVLB (M60) chassis	86.3
AVLB (M60) w/60 ft bridge	115.9

66. Material handling equipment. Forklifts, container handlers (such as the Rough Terrain Container Handler (RTCH-988B)), mobile cranes, straddle carriers, and tractors-with-trailers will be used, where possible, on piers/

wharves to move cargo and containers that have been temporarily stored on the wharf. Characteristics of these vehicles are presented in Appendix A.

67. Stacked containers. The loads produced by containers stacked on the wharf will depend upon the cargo being carried, the size of the container, and the number of containers stacked on top of each other. Assuming containers will not be stacked higher than three high, Table 3 gives some of the weights in pounds per square foot that can be expected from containers. All containers will not exert this much pressure on the deck since they will not be loaded to capacity at all times or with the heaviest classes of cargo. However, if they are loaded to their design capacity weight, as assumed in the table below, they will still not exceed 1,000 psf which is the design load for a number of new container ports. The 40-ft FLATRACK, which is designed to transport oversized loads that will not fit in a normal 40-ft container, can carry 67,200 lb of goods and should not exert a greater pressure on the pier than the normal 40-ft container.

Table 3

Deck Pressures from 20- and 40-ft Containers

Type	Stacked One High psf	Stacked Two High psf	Stacked Three High psf
20- × 8- × 8-ft MILVAN (44,800 lb)	280	560	840
20- × 8- × 8-ft Refrigerated MILVAN (40,320 lb)	252	504	756
40- × 8- × 8-ft Commercial container (67,200 lb)	210	420	630

68. Bulk or palletized cargo. The likelihood that all goods will be containerized is not certain. There are still a number of bulk carriers in the US fleet that may be used in time of war. These ships will still unload palletized bulk goods. Their weights are generally lower than containerized loads but in some instances can impart a larger load. Table 4 gives some general pressures of bulk cargos. Palletized loads can be stacked, and this

Table 4

Deck Pressures from Palletized Bulk Goods

Type	Stacked One High psf	Stacked Two High psf	Stacked Three High psf
Dry goods	60-150	120-300	180-450
Food stuffs	90-210	180-420	270-630
Portland cement	216-315	432-630	648-945
Bulk woods	135	270	405
Light hardware	900	1,800	2,700
Books	195	390	585
Wire*	425	850	1,275
Rope	100	200	300
Screws	300	600	900
Paper	105	210	315

* Only one roll high per pallet.

table presents deck pressures in pounds per square foot for pallets 4 × 4 ft containing 48 cu ft of goods for pallets stacked to three high.

Specific area loads

69. Cranes. Most larger ports around the world have been converted to container operations. To do this, the port authorities either replaced the older port structures or reinforced them to handle the increased weights imparted by containers and container handling cranes. To restore a damaged port to its container handling capability will require that crane operating areas and areas which will handle container off-loading and storage be reconstructed to the minimum design strengths for these conditions. If the weights imparted by the container cranes are not known, then the damaged areas must be repaired to the same structural condition as an undamaged area. It is difficult to anticipate the type of crane which will be located at an unknown port destination. There are many makes and types. The types of cranes which are likely to be found at any given port are as follows:

a. Jib.

b. Gantry.

c. Portal.

d. Container handling.

e. Level luffing.

f. Hammerhead.

g. Truck mounted.

The characteristics of these cranes are that they generally travel on two rails located parallel to the edge of the pier/wharf. Cranes generally have four legs with numerous sets of wheels on each leg (the minimum number of wheels per leg is four). Larger ports are usually equipped with the container handling crane, which is a large version of a portal crane. Smaller and older ports are equipped with other types of port cranes. The rail that is nearest to the edge of the pier will be the rail that receives the greatest loading. Rails will be less than 1/2 of 1 percent grade from one end of the pier to the other, since these cranes cannot climb a grade. The heaviest loadings will be exerted by container handling cranes. The data in Table 5 were compiled from manufacturers brochures, and in some places individual specifications were not available. However, these represent the range of loadings that can be expected for container handling cranes. For greater detail of the engineering of crane loads, refer to Naval Facilities Engineering Command manual NAVFAC DM-38.1 (Headquarters, Department of the Navy 1982).

70. Railroad loadings. With the acceptance of the container as the major means of moving goods through a port, the railroad tracks have been relocated from the pier/wharf to the warehouses. However, tracks are still prevalent at older ports, and they serve a valid purpose, especially in transporting grains and other bulk materials that do not conform to movement by container. Rail loadings are generally designed to Cooper's E-45 loading which consists of wheel loads, in pounds of two locomotives as shown in Figure 32, followed by a uniform load to represent a line of cars. The loading for the cars is 4,500 lb/lin ft. Table 6 gives reaction loads on supports as a result of Cooper's E-45 loading on various spans. Greater detail regarding this table and other engineering data relating to railroad loadings are presented in FM 5-35 (Headquarters, Department of the Army 1971) and NAVFAC DM-25 (Headquarters, Department of the Navy 1971).

Table 5

Weights and Loads of Various Container Cranes

Crane	Weight ton	Maximum wheel loads, ton Waterside	Landside
O & K BE 35	35	*	*
Liebherr B40	53-67	*	*
Liebherr LG 1400 truck crane	105.6	13.2 per each of 8 axles	
Liebherr 350L 50-ton crane w/gantry	159.3	35.3	35.3
Liebherr 500L 77-ton crane w/o gantry	168.0	*	*
Liebherr 750L 110-ton crane w/o gantry	204.0	*	*
Liebherr 1000L 110-ton crane w/o gantry	255.0	*	*
Liebherr 1500L 110-ton crane w/o gantry	320.0	*	*
Kocks 26-ton double guide crane	*	66.0	63.8
Kocks 28-ton level luffing portal crane	*	184.0**	102.3**
Kocks 45-ton container gantry crane	*	110.0**	*
Kocks 54-ton container transporter	*	363.0**	*
Wild 16.5-ton portal crane	60.0	16.5	16.5
Strother & Pitt 6-ton gantry	*	53.9**	*
Strother & Pitt 11-ton gantry	*	82.5**	*
Strother & Pitt 16.5-ton gantry	*	111.1**	*
Strother & Pitt 35-ton jib crane	*	36.0	36.0
Strother & Pitt 40-ton gantry	580.8	42.9	38.1
Paceco 30-ton container crane	*	7.0	7.0
Paceco 40-ton container crane	*	7.3	7.3

* Data not available.
** Corner load to be distributed over a number of wheels.

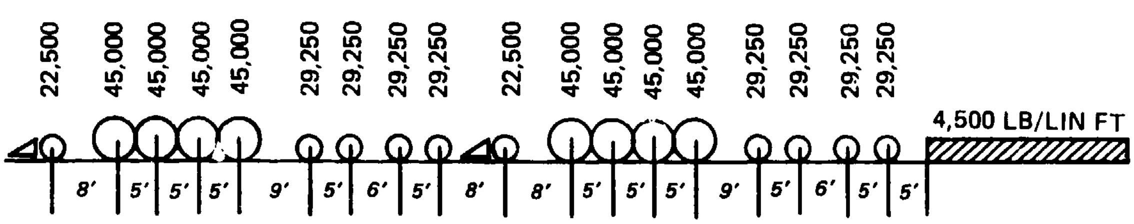


Figure 32. Cooper's E-45 loading

Table 6

Pier Reactions to Cooper's E-45 Loading

Span ft	Pier Reaction kips	Span ft	Pier Reaction kips
10	45.0	21	76.4
11	49.1	22	79.8
12	52.6	23	81.2
13	55.4	24	83.2
14	58.7	25	85.1
15	61.5	26	87.4
16	64.0	27	90.1
17	66.2	28	92.5
18	68.3	29	94.9
19	70.7	30	97.1
20	73.7	--	--

Container Traffic

71. Container ports consist of main pier/wharf areas which are dedicated to the uninterrupted flow of containers between shipside and inland transport modes. Container ships are usually discharged and backloaded at pierside in 24 to 48 hr as compared with 10 to 14 days for break-bulk ships. A container handling crane can normally operate on a 3-min cycle to pick up a container from a ship, relocate it on the dock or on a vehicle chassis, and be ready to pick up another container. Some container facilities estimate their capability by the hour and plan on a 20-container cycle per hour. A modern container terminal normally has access to rail and road transportation. Adequate space is required for container marshaling areas, with room for assembling containers for loading aboard ship or for holding containers being discharged and awaiting transport inland. Figure 33 illustrates the flow of containers through a typical container transfer facility. Detailed terminal operations are discussed in FM 55-70 (Headquarters, Department of the Army 1975a).

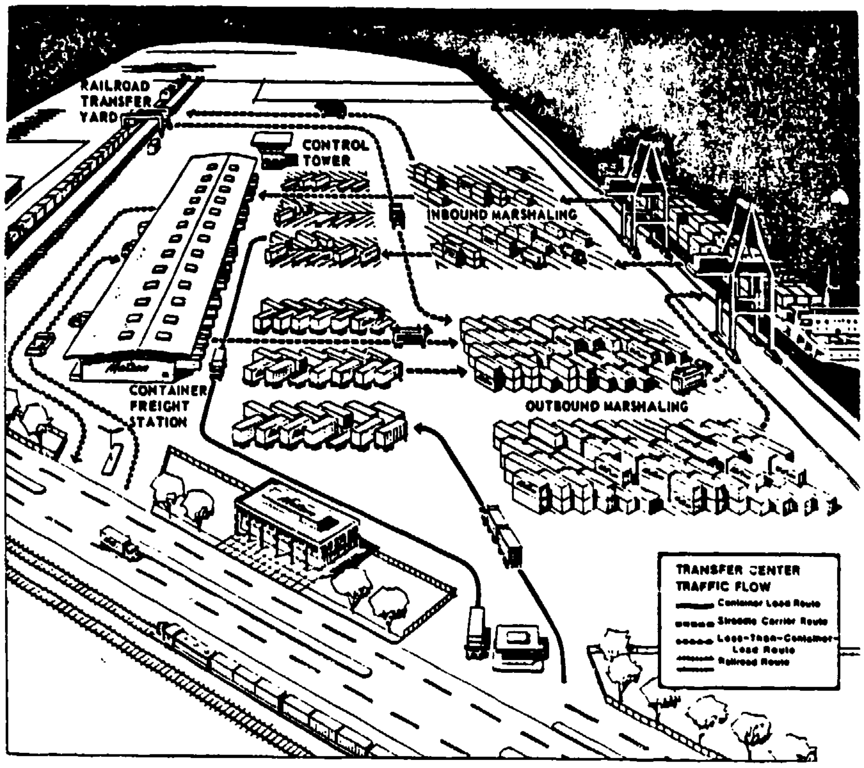
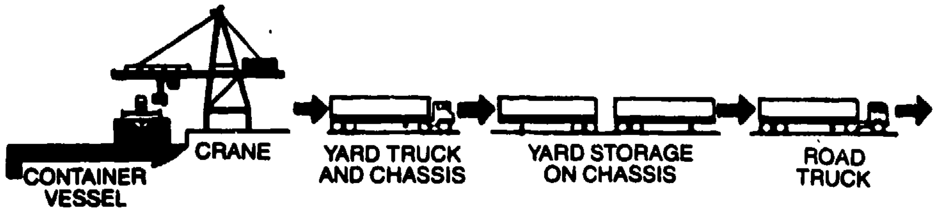


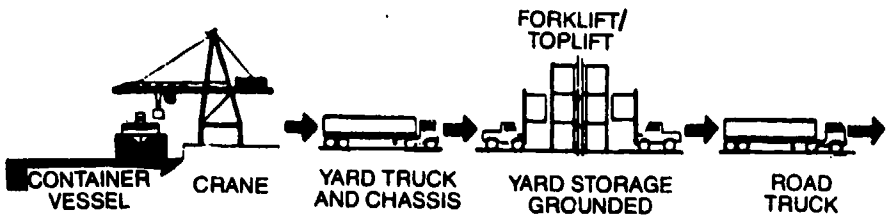
Figure 33. Example of a container transfer terminal and traffic flow

72. There have been various systems developed for moving containers through a container terminal quickly and efficiently. These systems can be classed as two basic types, wheeled and grounded. In a wheeled system, containers for inland movement are transferred from an ocean carrier to an over-the-road chassis, and chassis with container (on-chassis) are parked in the storage yard until a road truck arrives for inland transfer. This procedure is reversed for outgoing container traffic. Figure 34 shows the types of container movement systems. Figure 34a shows the flowchart of a typical chassis system. In the grounded system, containers are stored on the ground and stacked on top of one another. There are three basic methods for the yard storage operation: forklift/toplift system, travel crane system, and straddle carrier system. These systems are illustrated in Figures 34b, 34c, and 34d.

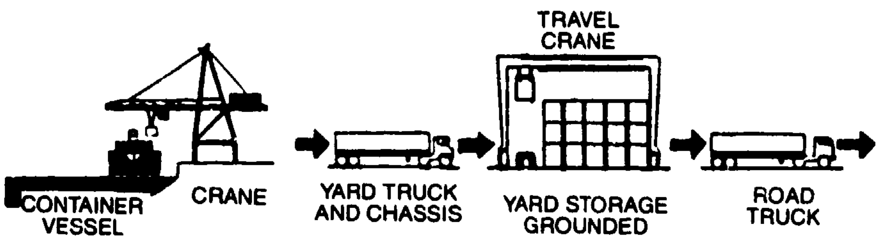
73. Determination of whether to stack containers or to store them on chassis in the storage yard is based on the space available for storage and availability of chassis/semitrailers. Because of the large number of containers which may enter the theater, there may be insufficient trailers to permit



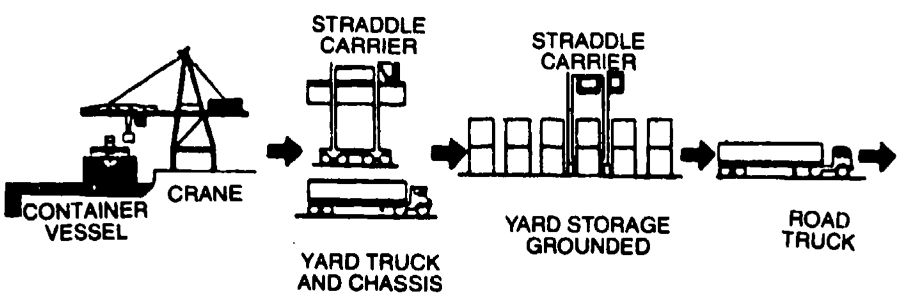
a. Chassis system



b. Forklift/toplift system



c. Travel crane system



d. Straddle carrier system

Figure 34. Types of container movement systems

on-chassis storage. However, if factors do permit on-chassis storage, container handling requirements are significantly reduced.

74. Commercial port operators and the military have developed a wide variety of material handling equipment for container movement in and around port terminals. Such equipment generally falls within two categories: that required for ship-to-pier transfer and that required for operations in the terminal area.

Ship-to-pier transfer

75. Container transfer is usually accomplished by gantry cranes available either as part of the ship's gear or as permanent pierside facilities. The trend has decreased from self-sustaining container ships; therefore, pier-mounted gantry cranes are used extensively for container transfer at pierside (Figure 35). Some ports use mobile or portable truck-mounted cranes in lieu of fixed gantry cranes (Figure 36).

Terminal area operations

76. Material handling equipment is needed at wharfside for container transfer to the storage area. Containers are generally transferred by tractor-trailers (Figure 37) or straddle carriers (Figure 38). After the container reaches its storage destination, containers are stacked by lifting devices such as straddle carriers, travel cranes (Figure 39), or forklifts/toplifts (Figure 40). The Army inventory includes container handlers such as the Rough Terrain Container Handler (RTCH) which is capable of operating as a rough terrain vehicle in storage and marshaling areas (Figure 41). During military contingencies involving war-damaged port operations, military personnel may be faced with situations whereby specific container handling equipment is not available to transfer containers from dockside to storage areas. Caution should be exercised not to traffic piers/wharves with vehicles which will overload these port facilities. Container pier/wharf designs should be investigated before mobile cranes or container handling forklifts are used in lieu of war-damaged fixed cranes or lack of tractor-trailers and straddle carriers.

Pile Repair/Replacement

77. The pile supporting structure of piers/wharves is a critical element when repairing these war-damaged structures, since support structures will require repair or replacement before the decks can be repaired and

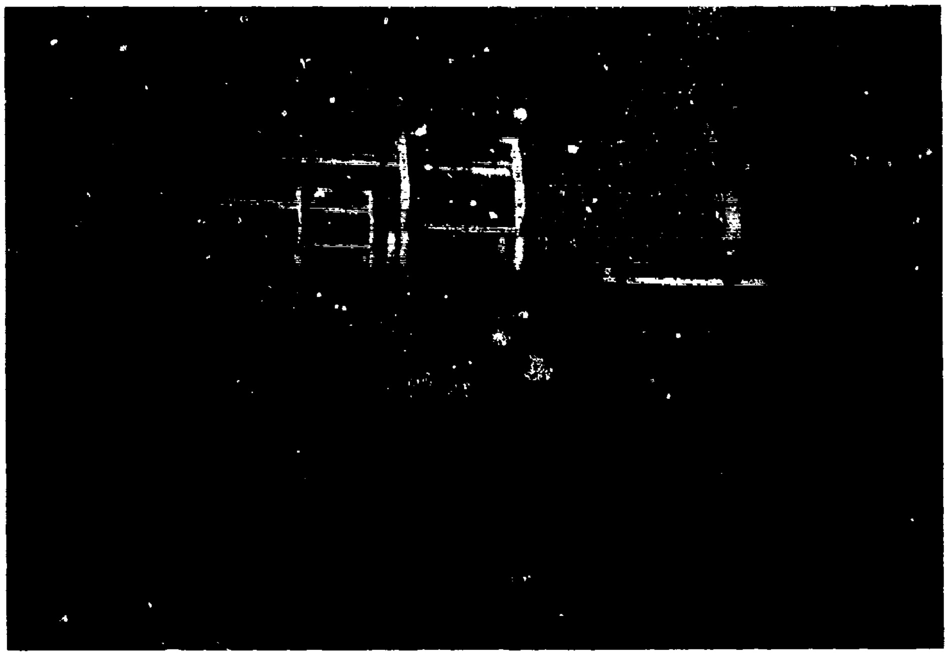


Figure 35. View of a gantry crane at Port of Oakland

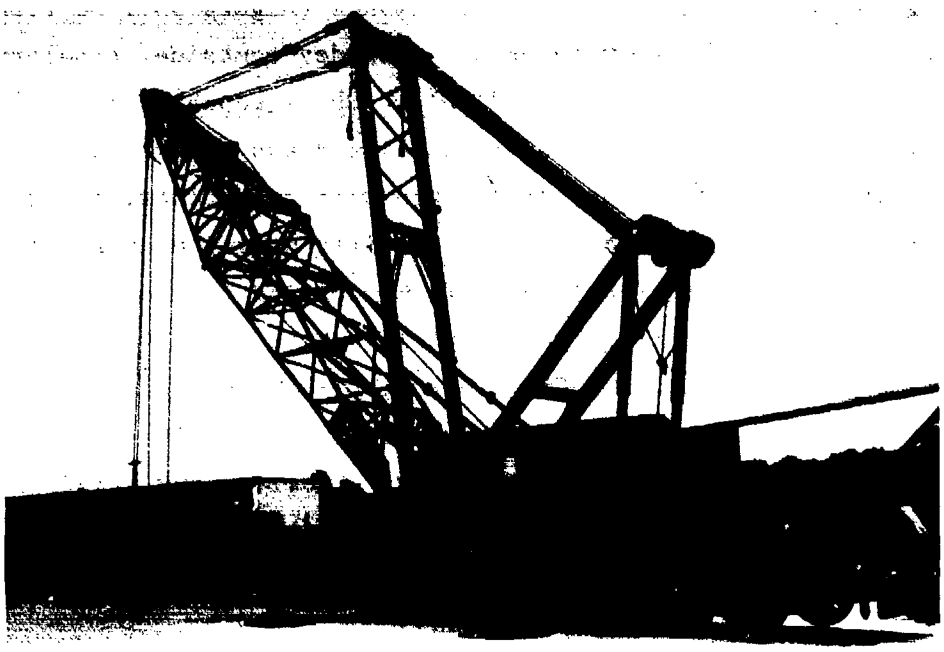


Figure 36. View of a 250-ton mobile crane

Figure 37. Container transfer from wharfside by tractor-trailer at Port of New Orleans

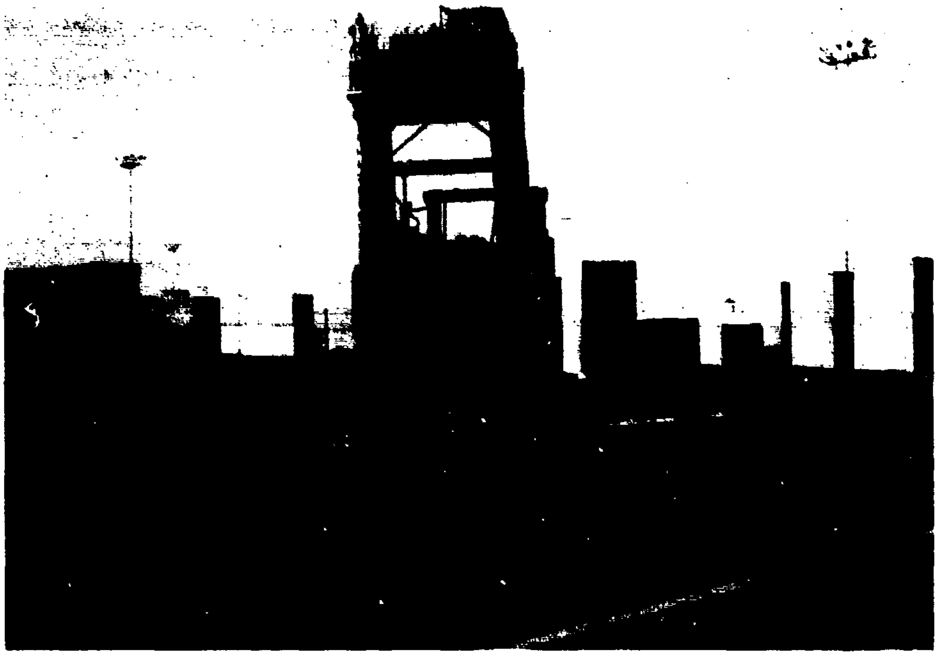


Figure 38. Container transfer to storage area by straddle carrier at Port of Oakland

Figure 39. Travel crane (rubber tired) used in storage area to stack containers at Port of Oakland

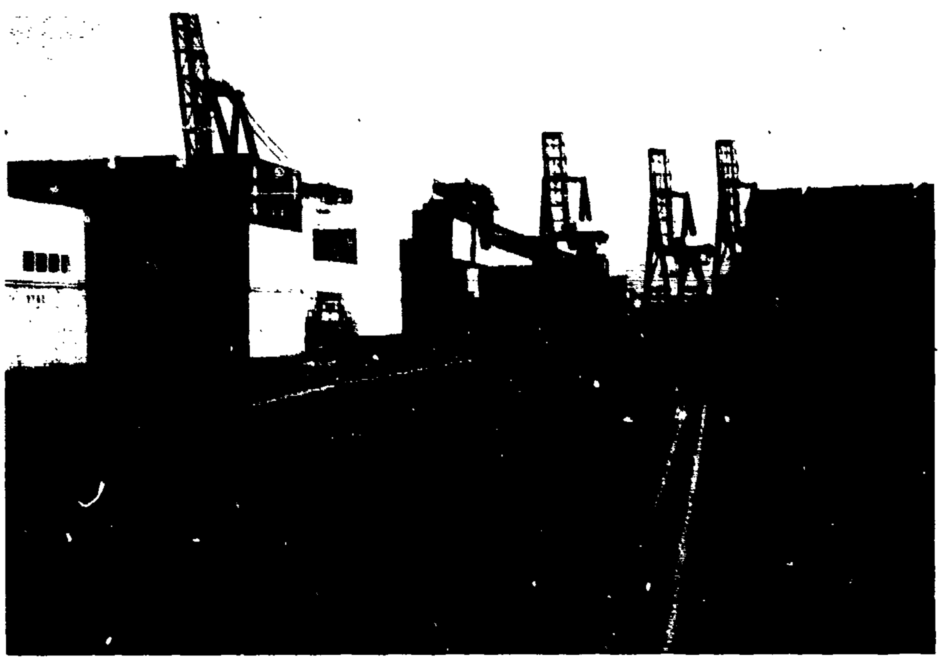


Figure 40. Forklift/toplift used in storage area to stack containers at Port of Oakland

Figure 41. View of RTCH

subsequently used. Decisions on whether to repair or replace piling are dependent on numerous variables and will have to be assessed on an individual basis. Several questions can be asked that will assist in evaluating whether the piling should be replaced.

a. Is the pile repairable? For example, is it split lengthwise? Has it been extensively damaged by marine borers? Is it severely splintered?

b. Are materials readily available to permit pile replacement?

c. Is the pile stub accessible, and can it be removed with the available equipment?

d. Are a pile driver and properly trained operator available to drive the replacement pile?

e. Will the type of repair considered support the anticipated load?

f. Do operational requirements permit the time required for pile replacement?

A report by Eastport International, Inc. (1986) presents extensive underwater repair discussions which include expedient repair concepts for foundations, support structures, and decking to support interface. The following information is presented should a decision be made to replace a pile rather than repair the damaged piling.

Pile replacement

78. The replacement of damaged piles involves removing any deck or substructure material that would interfere with the removal operation, withdrawing the damaged piles, and driving new piles to take their place. This is a very time-consuming operation and requires a typical pile driving crew of seven personnel. It is most likely that piles will be removed only if they are physically in the way or would cause more problems if they were not removed. If it is necessary to provide new piles to carry a foundation load, it is much more desirable to drive new piles next to the damaged ones or repair the damaged piles thereby eliminating the time, expense, and labor necessary to withdraw the damaged piles.

79. Should it be necessary to extract a pile or series of piles, the following equipment may be used.

- a. Block and tackle assembly.
- b. Vibratory driver/extractor hammer.
- c. Hydraulic jacks.
- d. Driver/extractor barge.
- e. Work boats.

The amount of time it takes to extract a pile will vary, depending on the sophistication of the equipment used. For instance, if the pile can be removed with a vibratory driver/extractor, the time and effort will be significantly less than that using only a block and tackle. Work will also be slower if it must progress from the deck of a barge instead of from a stable platform such as the dock. Once the piles have been extracted they can sometimes be easily replaced by new piles. The replacement pile can be driven in the hole made by extracting the damaged pile. The pile of similar size and shape should be driven until refusal or to the specified number of blows per foot if it is a friction pile. The damaged pile that is removed should be retained and the damaged portion removed. The remainder may be used at another repair location.

80. Driving of new piles next to damaged ones is a more desirable method of pile replacement. There are several methods which can be used to drive these piles. If the pier/wharf structure can withstand the loads of a pile driver in the vicinity of the damaged area, then a crawler-mounted or skid-mounted driver positioned on the deck is the preferable method of driving

new piles next to the old ones. If the damaged area is too large for a deck supported pile driver to get close enough, then a barge-mounted driver is required. Replacement of a damaged timber pile with a new one in a situation where the deck or pile cap does not need repair is performed by driving a pile next to the damaged pile. It will be necessary to cut a hole in the deck next to the damaged pile through which the new pile can be driven. The new pile is inserted through this hole and driven into the bottom directly adjacent to the old, damaged pile. The new pile may have to be driven at a small angle to get the tip of the pile to penetrate the bottom directly beneath the pile cap. When the pile has been driven, workers move beneath the deck and cut off the new pile such that it may be pulled into place beneath the pile cap. This can be done with a portable winch or other methods of pulling the pile into place. Pile alignment techniques are discussed in TM 5-360 (Headquarters, Department of the Army 1964). Shims must be driven between the pile and cap in order to ensure that transfer of loads will take place. The pile is then fastened to the pile cap with a drift pin, and the hole in the deck is ready for repair.

81. The tools required for pile repair are a pile driver, cutting equipment to produce a hole in the deck, and a hand winch. A pile driving crew and workers to cut and repair the hole in the deck are also needed. Additional methods of pile driving are discussed in FM 5-134 (Headquarters, Department of the Army 1985b).

82. Driving piles by jetting is another method of considerable value when the soil conditions resist driving by conventional methods. Jetting is the process of forcing water under high pressure around the tip of a pile being driven into the soil. This is accomplished by lashing one or two water pipes on each side of the pile with the pipe nozzle near the pile tip as shown in Figure 42. The high-pressure water being jetted out of the nozzle disturbs the soil at the tip of the pile and allows the water to penetrate the soil with little or no assistance from a hammer. The pile is jetted into the soil until it is within a few feet of the final elevation and then the jetting is discontinued. The final driving is performed by a hammer. This method may be beneficial in soils where heavy hammering may further damage pier structure near the driving operation or in situations where hammer driving may damage the replacement pile. As shown in Figure 42, the orientation of the pile during the driving operation is maintained by using ropes attached to the head of the pile. Since the soil is disturbed by the jetting, the pile will not

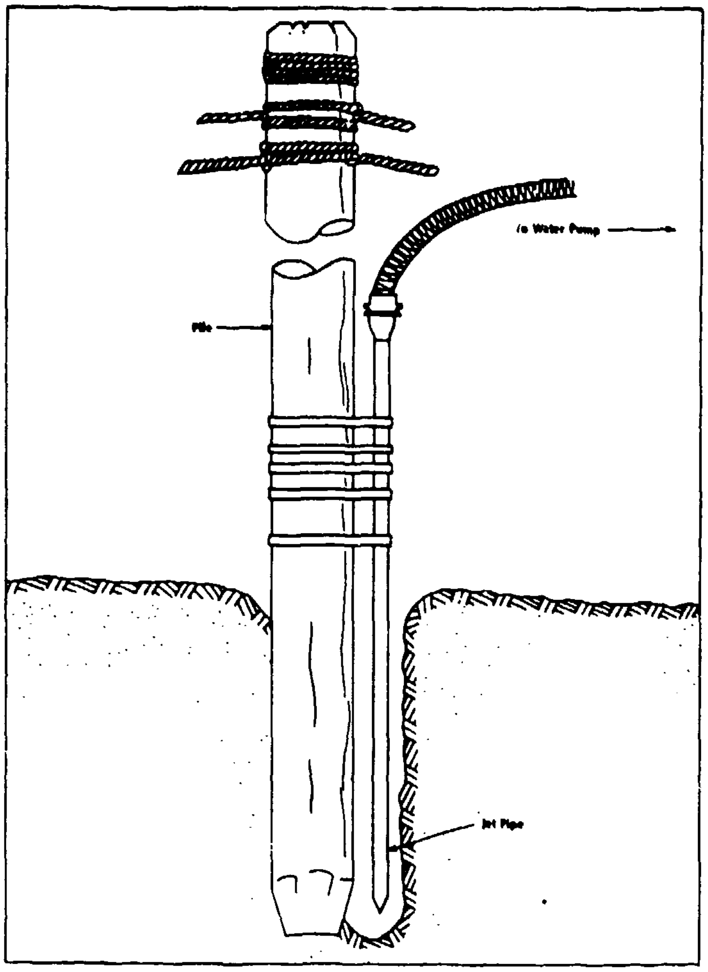


Figure 42. High-pressure water jetting of pile

remain vertical without assistance. This method does not affect the ultimate load-carrying capacity of the pile since the disturbed soil will settle around the pile within a day or two and provide the support that is required. The tools required for pile jetting are an air compressor, pipe and hosing, crane, block and tackle to guide the pile, and a crew of five personnel (three to guide the pile, a crane operator, and a compressor operator).

83. Jetting can also be performed to install precast concrete piles. If the piles are of the solid variety, then similar techniques presented in previous paragraphs using the water jet pipes can be used. If the concrete piles have been designed with internal passages to allow the jetting water to pass down the center of the pile and exit at the tip, then the external jet pipes are not needed (Figure 43).

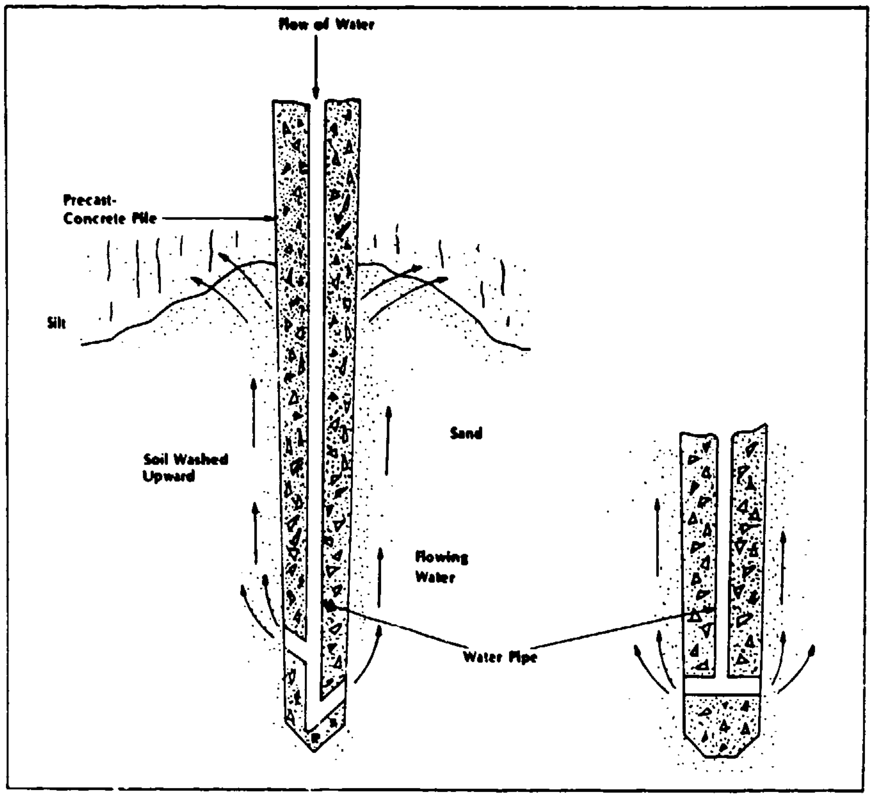


Figure 43. Precast concrete piles with internal jetting pipes

Concrete Repair of Decking

84. One method to repair damaged piers/wharves which can also be utilized to improve the load-carrying capacity of the deck is to replace the deck structure with cast-in-place (CIP) concrete. Damage to timber as well as concrete structures can be repaired by using CIP concrete. Timber structures can also be strengthened with CIP concrete overlays which improve the load-carrying capacity of the deck. This fact alone is important when trying to increase the load-carrying capacity of the structure to support containerized cargo and handling equipment. Discussions for replacement of damaged decking with cast-in-place concrete are presented in Appendix B. Repair of damaged piers/wharves with concrete is only viable when concrete equipment and materials or host nation support is available.

85. Another method of repairing damaged piers/wharves is to replace the deck structure with precast concrete structural elements. Discussions of the precasting process are presented in Appendix C. Another possibility for

repairing port structures with precast concrete elements is to preposition critical structural elements near the port complex. This would eliminate the time involved to implement precasting operations for fabricating structural elements.

Concrete Repair Techniques

Introduction

86. One of the most important steps in the repair or rehabilitation of a concrete structure is the preparation of the surface to be repaired. The repair will only be as good as the surface preparation regardless of the nature or sophistication of the repair material. For repairing concrete surfaces, the surface of the concrete that remains after a bomb attack must be properly prepared. Repairs of reinforced concrete must include proper preparation of the reinforcing steel. Proper steel preparation develops bond with the replacement concrete and ensures desired behavior of the structure.

Preparation of concrete surfaces

87. Surface preparation includes all steps taken after removal of large volumes of concrete by blasting or other methods as well as steps taken to prepare surfaces when little or no concrete is removed. The same techniques are used in both instances.

88. In most concrete repair projects, the zones of damaged concrete are not well defined. Most references state that all damaged or deteriorated material should be removed. However, it is not always easy to determine when all such material has been removed. Thus the best recommendation is to continue to remove material until aggregate particles are being broken rather than simply removed from the cement matrix.

89. Removal of concrete using blasting or other violent methods may cause damage to the concrete remaining in place. Past experience using these techniques have indicated that thin layers, which have become delaminated from the blasted surface, remain in place after such a blast. These areas can be identified by the use of a hammer to take soundings. All such layers must be removed before replacement.

90. When concrete is removed using impact tools, there is the potential for very small-scale damage to the surface of the concrete left in place. Unless this damaged layer is removed, the replacement material will suffer and

will appear as a bond failure. If this happens, a perfectly sound and acceptable replacement material may fail due to improper surface preparation. In all cases in which concrete has been removed from a structure by a primary means such as blasting, impacting, or splitting, the concrete remaining should also be prepared using a secondary method such as wet sandblasting or high-pressure water jetting to remove any remaining damaged surface material.

91. If the areas to be repaired are small, preparation of the perimeter of the repair area is required. This is to provide a minimum thickness of repair material at the edge of the repair (i.e., to avoid feather edges). In some repair techniques it may be desirable to undercut the repair area (dry packing); in others, it is not desirable to undercut (shotcrete).

Methods of surface preparation

92. Chemical cleaning. Contaminants such as oil, grease, or dirt must be removed from concrete surfaces prior to placing repair materials. Detergents, trisodium phosphate, and various proprietary concrete cleaners are available for this work. It is also important that all cleaning agents be removed after the contaminated material is removed. Solvents should not be used to clean concrete since they will dissolve the contaminant and penetrate deeper into the concrete. Muriatic acid, commonly used to etch concrete surfaces, is relatively ineffective for removing grease or oil.

93. Mechanical cleaners. There are a variety of mechanical devices available for cleaning concrete surfaces. These devices are suitable for preparing surfaces for overlays or protective coatings. Included in this category are impact tools (scabblers), grinders, and impact abrasion tools. Depending on the head type of the tool, or the abrasive material used, a variety of surface preparation degrees may be achieved. All of these tools are not available from Army supplies.

94. Blast cleaning. Both wet and dry blast cleaning and water jet cleaning are used to prepare concrete surfaces for receiving new material. Due to dust and health problems associated with dry sandblasting, it is not frequently used or recommended. When using sand-blasting equipment, the air source must be equipped with an effective oil trap to prevent contaminating the concrete surface during the cleaning operation. Water jet cleaning equipment with operating pressures up to 20,000 psi is recommended for this type of surface preparation.

95. Acid etching. Acid etching of concrete surfaces has long been used to remove weak, surface layers of cement and normal amounts of dirt. The acid will remove enough cement paste to provide a roughened surface and thus improve the bond for replacement materials. American Concrete Institute (ACI) Committee 515 recommends acid etching only be used when no alternative methods of surface preparation can be used. The preparation methods described earlier are believed more effective than acid treatment. If acid is used, the following procedure is normally followed:

a. The surface should be cleaned of grease and oil using appropriate agents. The cleaning agents should be rinsed from the surface before the acid is added.

b. Acid is then added at a rate of 1 qt/sq yd, and it should be worked into the concrete surface with a stiff brush or broom.

c. When the foaming stops (3 to 5 min), the acid should be washed off. Brooms should be used to remove reaction products and loosened particles.

d. The surface should be checked with litmus or pH paper to determine that all acid has been removed.

Final surface treatment

96. The desired concrete surface condition immediately before repair depends upon the type of repair being undertaken. The surface should be clean, dry, and dustfree.

97. The use of a bonding agent for repairs is open to question. General agreement does not appear within various concrete literature. The best general guidance is that small thin patches should receive a bonding coat while larger replacements probably do not require any bonding agent. Excellent bond of fresh to hardened concrete can be achieved with proper surface preparation and without the use of any bonding agents.

98. If bonding agents are used, several agent types are available. The most common are grout mixtures of portland cement, fine aggregate, and water. The water-cement ratio of the grout should be approximately the same as that of the replacement concrete, and the grout should have a consistency of a thick cream. The grout must be worked into the surface using a stiff broom or brushes. The grout should not be allowed to dry before the concrete is placed.

99. There is a wide variety of epoxy and other polymer bonding agents available. If one of these products is used, the manufacturer's

recommendations must be followed very closely or a bond breaker rather than a bond enhancer may result.

Preparation of the reinforcing steel

100. Preparation of the reinforcing steel in the repair or replacement of any reinforced concrete member is very important. The surface condition of the steel is crucial for the cement to bond with the steel. In the case of repair to war-damaged concrete structures, there will be many situations where the reinforcing steel will be damaged, and it will be necessary to determine what needs to be repaired or replaced.

101. The determination of whether to repair or replace will include how severely the steel has been stressed, whether it has suffered heat damage, if it has been disrupted from the concrete so badly that the bond between it and the concrete has broken, and several other considerations. For instance, in determining the extent of reinforcement damage, it may be determined that the steel has been deteriorated by sea water and has become corroded over the years. The decision to repair or replace must be made by an engineer who is competent in structural reinforcing design practices.

102. Once the cause and magnitude of the reinforcement problem has been determined, the steel must be exposed, the structural condition evaluated, and the undamaged steel prepared for the repair techniques. Proper steps in preparing the reinforcement will ensure that the repair becomes a permanent solution rather than a temporary one that will deteriorate in a short period of time.

103. Although this section is primarily concerned with the preparation for repair to reinforcing steel, several statements should be devoted to the care of removing the surrounding deteriorated concrete. Most frequently, the deteriorated concrete will be removed with a jack hammer. For this purpose, a 30-lb hammer should be sufficient and will not significantly damage the sound concrete at the periphery of the damaged area. Extreme care should be exercised to ensure that further damage to the reinforcing is not inflicted in the process of removing the deteriorated concrete. Jack hammers can heavily damage reinforcing steel if the hammer is used without knowledge of the location of the steel hidden beneath the damaged concrete. For this reason, a pachometer should be used to determine the depth and location of the steel in the concrete. Also, if available, structural drawings of the pier/wharf should be obtained to determine the location of the reinforcing steel.

104. After the larger pieces of concrete have been removed, a smaller chipping hammer should be used to remove the concrete near the reinforcement. All the weak, damaged, and easily removable concrete should be chipped away. A good, strong repaired area placed over a supposed sound concrete can become a further repair problem when weak concrete that appears to be of good quality breaks away from the structure. For this reason, it is important to remove concrete to a depth of quality concrete material.

105. In order to determine the full damage to reinforcement steel, the concrete should be completely removed from around the bar to allow 1/4 in. clearance between the surface of the steel and the concrete plus the dimension of the maximum size of aggregate to be used in the repair. This not only allows complete inspection of the steel, it also means the repair patch will completely surround the steel and will be anchored to the substrate more securely.

106. The reinforcement should be inspected for damage wherever the concrete has been removed. At this point a decision whether to remove and replace the reinforcement must be made. Before any decision is made, all loose debris such as pieces of cement, rust, charring from fire, etc. must be removed to make an accurate assessment of the condition of the steel. The services of a structural engineer should be used to determine whether or not the reinforcement should be replaced. The engineer's judgment and knowledge of the purpose of the reinforcement should be used to decide whether to repair or replace.

107. The easiest method of replacing reinforcement is to cut out the damaged area and weld in replacement bars. If welding is used, the splice should be a lap splice with a welded overlap of 10-bar diameters on each end of the spliced area. Butt welding should be avoided due to the high degree of skill required to develop a full penetration weld. Also, high strength steel should not be welded because of the damage inflicted to the steel during the welding process. Welding must conform to the structural welding code (American Welding Society 1984) for reinforcing steel. Where welding is not used, a conventional lap splice should be used. The requirements for length of conventional lap splice should conform to the requirements of ACI 318-83 (American Concrete Institute 1984a) for splice length of reinforcement.

108. If it is determined that the steel does not need replacing, the steel should be thoroughly cleaned of all rust and foreign matter before the

new concrete material is placed. The steel should be sandblasted to clean metal. Sandblasting is the preferred method of cleaning because it not only removes any rust from the surface of the steel but also removes loose particles of mortar which may have been stuck between aggregate particles. Rust and loose particle removal will ensure a better bond between new concrete and the existing substrate. Effort should be made to ensure that sandblasting has removed all the rust from the underside of the reinforcing bar, particularly in situations where the concrete has only been removed to expose just the reinforcing bar. The underside is not directly hit by the high pressure sand particles and must rely on rebound force from the substrate concrete surface.

109. The air compressor used in conjunction with sandblasting is important. After cleaning the steel and blowing loose particles from the repair area, it is important that neither the reinforcing steel nor the concrete substrate surface be contaminated with oil from the compressor. Either an oil-free compressor or one that has a good oil trap should be used.

110. An alternative method of cleaning the steel is high-pressure water jet cleaning. Compared to sandblasting, this method provides the water and oxygen necessary to begin the corrosion process again once the steel has been cleaned. If water jet cleaning must be used, the steel must be dried and protected immediately after cleaning.

111. After the steel has been cleaned, workers should ensure the steel is correctly located before concrete is placed. If the steel has been bent due to the force of a bomb blast, it should be straightened and placed in its original location. Field bending should not be performed without authorization of the inspecting engineer. He must specify whether the bars should be bent cold or if heating should be used.

112. Tests have shown that most A615 Grade 40 and Grade 60 reinforcing bars can be bent and straightened, preferably about the weak axis, up to 90 deg. If cracking or breakage is encountered, heating to a maximum temperature of 1,500° F would be beneficial. Minor cracks in the bend region, less than about 0.010 in. wide, should not adversely affect the performance of the bar.

113. Heating must be performed in a manner that will avoid damage to the concrete. If the bend area is within 6 in. of the concrete, some protective insulation should be applied. Heating of the bar should be controlled by temperature indicative crayons or other suitable means. The heated bars

should not be artificially cooled (such as by water or forced air) until after cooling to at least 600° F. Bars that fracture during bending or straightening can be spliced outside the bend region. Two things should be considered when bending reinforcing steel: (a) breakage and cracking is likely when straightening is performed at cold temperatures and (b) bars should be straightened with a slowly applied force rather than by hammering.

114. If new reinforcement is added to existing reinforcing, care should be taken when preparing the existing reinforcement because it is probably still carrying load. Cutting of any steel should be accomplished in a safe manner so as not to injure any workers should the steel whip or snap upon being cut. If possible, leave the reinforcing in place. Also, if the load on the structure is not relieved during the repairs, new reinforcement added to the structure will carry a lesser stress than the existing bars, and as a result the concrete will have to carry a larger part of the stress.

115. When it is necessary to install formwork around the area to be concreted, the construction team should make sure that the steel is properly distanced from the formwork to allow for minimum cover of concrete over the steel, particularly where the concrete is in a salt-water environment. Care should be exercised to ensure that no oil gets on the steel before the concrete is placed.

116. Before concreting, the steel can be further prepared by painting it with a cement slurry coating to enhance the bonding with the repair concrete, and to prevent any further deterioration before the repair is made. An alkaline cement slurry or an epoxy coating can be brushed onto the steel to coat it and prevent rusting until the repair is complete. The cement slurry should consist of portland cement and fine sand and just enough water to make the slurry about the consistency of thick cream.

117. After the prior information of reinforcement preparation have been followed, the repair area is ready for the replacement concrete. If good preparation practices are followed, a good repair job will result and the likelihood that it will have to be repaired again is minimal unless further damage occurs.

Cast-in-Place Repair Techniques

Introduction

118. This section contains descriptions of various methods and materials that can be used to repair concrete members that have been damaged, but can be salvaged rather than replaced. Each entry will include a description, application, procedure, and equipment necessary to complete the procedure. These methods are intended to restore the structure back to normal use, but in some cases are only temporary solutions to the repair problem.

Additional reinforcement

119. Description. Additional reinforcement is the provision of additional reinforcing steel, either conventional reinforcement or prestressing steel, to repair a cracked concrete section or improve the strength of an undamaged one. The added steel is to carry the tensile forces of the member either where the member has cracked or where additional tensile force is expected.

120. Application. Additional reinforcement can be used to repair cracked members or to increase the strength of an unbroken member. Particular applications relating to pier deck repair are also applicable to pile caps, stringers, girders, facia beams, deck panels, or any other concrete member that has sufficient depth and area in the vicinity of the cracked surface to utilize the technique.

121. Procedure.

a. Conventional reinforcement. This technique consists of sealing the crack, drilling 3/4 in. diam holes across the crack at 90 deg to the crack plane as shown in Figure 44, filling the holes and crack plane with epoxy pumped under low pressure (50 to 80 psi), and placing a reinforcing bar into the drilled holes (Stratton, Alexander, and Nolting 1982). Typically, No. 4 or 5 bars are used to extend at least 18 in. on each side of the crack. If possible, the number of bars used should be sufficient to extend beyond the extremities of the crack. The epoxy bonds the bar to the walls of the hole, fills the crack plane, bonds the cracked concrete surfaces together in one monolithic form, and thus reinforces the section.

b. A temporary elastic crack sealant is required for a successful repair. Gel-type epoxy crack sealants work very well within their elastic limits. Silicone or elastomeric sealants work well and are especially attractive in cold weather or when time is limited. The sealant should be applied in a uniform

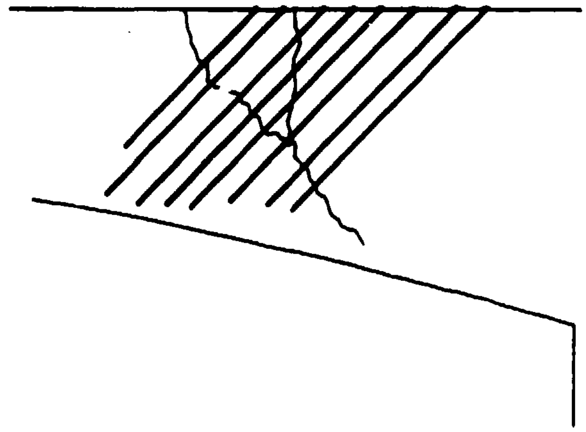


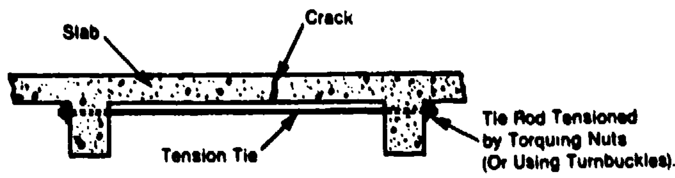
Figure 44. Crack repair using conventional reinforcement with drillholes 90 deg to the crack plane

layer approximately 1/16 to 3/32 in. thick and should span the crack by at least 3/4 in. on each side. The epoxy used to rebond the crack should conform to ASTM C 881-78 (American Society for Testing and Materials 1978), Type I, low-viscosity grade.

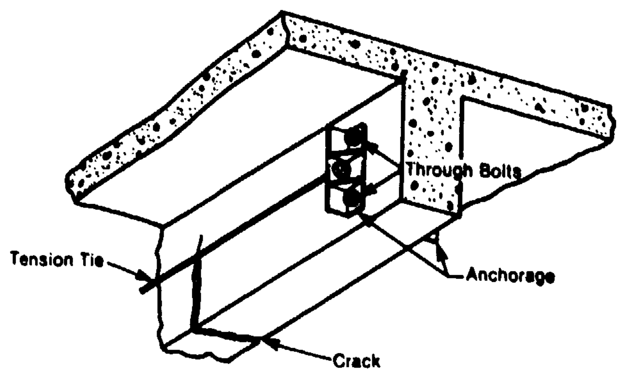
c. Reinforcing bars should be spaced to suit the needs of the repair. They can be placed in any desired pattern, depending on the design criteria and the location of the in-place reinforcement.

d. Prestressing steel. This technique uses prestressing strands or bars to apply a compressive force to the concrete in the area which has been cracked. This relieves the tension that initially caused the cracking. One form of this technique is shown in Figure 45. Adequate anchorages must be provided for the prestressing steel, and care is required so the problem will not migrate to another part of the structure. The effects of the tensioning force (including eccentricity) on the stress within the structure should be carefully analyzed. For indeterminate structures, the effects of secondary moments and induced reactions should be regarded if posttensioning is being considered. The procedure can be used to repair slabs as shown in Figure 45a or beams as shown in Figure 45b. If the prestressing steel is being used as in Figure 45b, then a bolt needs to be anchored into or through the beam. The prestressing can be performed on the side of the beam as shown in the figure or beneath the beam.

e. Posttensioning. Another method of closing tensile cracks by prestressing techniques is to drill holes across the crack as mentioned in paragraph 121a and anchor a prestressing bar on the side of the crack opposite from where the hole was drilled. Then, by tensioning the bar against the concrete, the crack can be pulled closed. The tensile stress on the steel can be held in place by a nut or chuck bearing on the



a. To correct cracking of slab



b. To correct cracking of beam

Figure 45. Crack repair using external prestressing strands or bars to apply a compressive force

concrete surface. The bar should be threaded on the near-end and long enough for the threaded part to protrude through the beam where it is tensioned. Anchoring on the far side of the crack is achieved by placing an epoxy "sausage" cartridge in the bottom of the hole and spinning the stressing bar into the cartridge. The cartridge consists of an epoxy resin and a strip of hardener encased in a light plastic casing in the shape of a sausage about 1 ft long. The hardener is in contact with the epoxy, but they have not been mixed. Both remain separate components. Spinning the prestressing rod into the cartridge breaks the plastic and mixes the epoxy and hardener together and around the bar. The epoxy then hardens and anchors the rod to the bottom of the hole. The hole should be about 1/8 in. in diameter larger than the reinforcing rod. The number of cartridge sausages used to anchor the reinforcing bar to the walls of the hole will depend on the size of the cartridge and the volume of the hole after the reinforcing bar is inserted. The number of cartridges used should be based on the manufacturers' recommendations. The hole should be drilled at least 18 in. beyond the plane of the crack to allow for adequate length of bonding.

f. Stitching. This method involves drilling holes on both sides of the crack and grouting in stitching dogs (U-shaped metal units with short legs) that span the crack (Johnson 1965). As shown in Figure 46, the stitching dogs are grouted into the

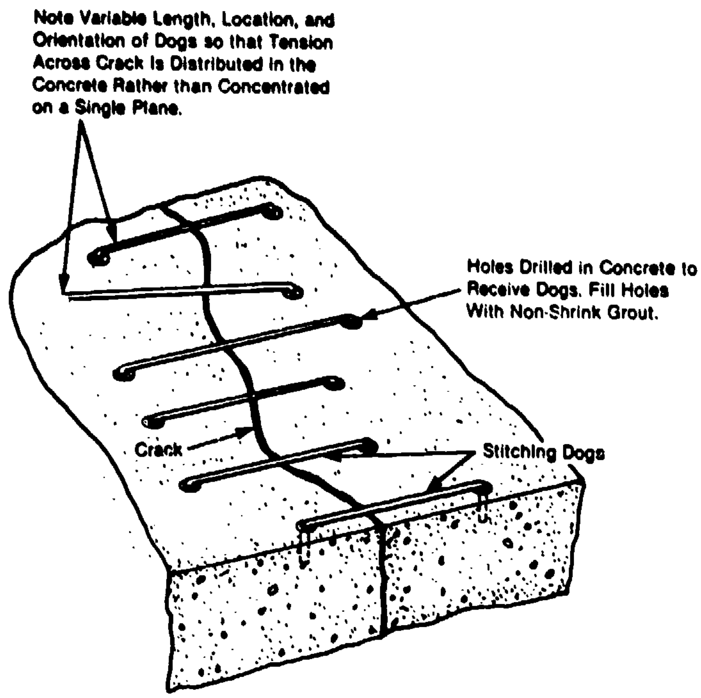


Figure 46. Repair of a crack by stitching

good quality concrete on either side of the crack using either a nonshrink grout or an epoxy-resin-based bonding system. The stitching dogs should be variable in length, and their orientation should be located so that the tension transmitted across the crack is not applied to a single plane within the section but is spread over an area.

g. Spacing of the stitching dogs should be reduced at the end of cracks. In addition, consideration should be given to drilling a hole at each end of the crack to blunt it and relieve the concentration of stress as discussed in ACI 104-71 (American Concrete Institute 1982).

h. Where possible, both sides of the concrete section should be stitched so that further movement of the structure will not pry or bend the dogs. In bending members, it is possible to stitch one side of the crack only. This should be performed on the tension face where movement will occur. If the member is in a state of axial tension, then the dogs must be placed symmetrically, even if excavation or demolition is required to gain access to the opposite sides of the section.

i. Stitching will not close a crack but can prevent it from propagation. If water is a problem, the crack should be made watertight as well as stitched to protect the dogs from corrosion. Waterproofing should be completed before stitching begins. For active cracks, the flexible sealing method may be used in conjunction with the stitching techniques.

j. The dogs are relatively thin and long and cannot take high compressive force. Accordingly, if there is a tendency for the crack to close as well as to open, the dogs must be stiffened and strengthened by encasement in an overlay.

122. Necessary equipment. Each of these methods requires the drilling of the damaged concrete. It will be necessary to have at least one drill and a number of masonry drill bits of sufficient length to extend beyond the plane of the crack. The external prestressing method requires the length of the drill bits to be only long enough to provide the proper anchorage for the prestressing rod. That length may be the thickness of a concrete beam, or deep enough into a member to anchor bolts that will tie a steel plate to the concrete. To prestress by tightening a nut against an anchor plate, a large wrench will be necessary. Where the reinforcing will be held in the member by filling the holes and the crack with epoxy, pumping equipment capable of providing up to 100 psi pressure will be necessary. If the crack is very wide and has caused some deflection or sagging of other members, then jacking equipment to close the crack and return the damaged concrete to grade will also be necessary. The bulk of these procedures can be accomplished by a driller and two helpers to accomplish the addition and tensioning of the reinforcement.

Conventional concrete placement

123. Description. This method consists of replacing the defective concrete with a new conventional concrete mixture of suitable proportions that will become an integral part of the base concrete.

124. Applications. If the defects in a structure go entirely through a wall or slab or if the defects extend deeper than the level of the reinforcement, and if the repair area is large, then the replacement of the concrete is desired.

125. Procedures. The two procedures that are generally used to replace concrete members with other concrete members are cast-in-place concrete and precast panels. Cast-in-place concrete techniques are described in detail in Appendix B, and details of precast techniques are given in Appendix C.

Shotcrete

126. Description. Shotcrete is defined as "mortar or concrete pneumatically projected at high velocity onto a surface..." (ACI 1982). It is a form of mortar or concrete which is conventional in its composition but

applied to the surface in a manner which is very different from conventional placement techniques. Shotcrete can be prepared for a dry or wet mix process and is applied to its final surface by being sprayed onto the formwork. This process allows the concrete, which is usually dry compared to conventionally placed concrete mixtures, to be applied to horizontal, vertical, and even overhead surfaces due to the cohesiveness of its matrix.

127. The properties of shotcrete depend on the mixture design, the conditions under which it is placed into the formwork, the type of delivery process, and the expertise of the placement crew. Normally, shotcrete is designed to be a stiff cohesive mixture to adhere to vertical and overhead surfaces; however, it can be altered through the use of admixtures to have other properties which may be desirable for placement in awkward locations. The type of equipment used to place the shotcrete can make a difference in its final properties. The following paragraphs describe the equipment and their effects. The equipment required to place shotcrete include a compressor, hose from the compressor, equipment to batch the wet or dry materials, and a pneumatic "gun" from which the shotcrete is projected. Probably the most critical ingredient in good properties of shotcrete is the technique that the gun man uses to apply the mixture to the formwork.

128. Applications. Shotcrete can be used as a repair material to build up or repair a structural member or as a construction material in areas where structures must be completely rebuilt. As a repair material it can be applied to damaged surfaces such as concrete slabs, walls, beams, columns, and piles to rebuild the damaged area to the proper thickness and replenish its load-carrying capacity. It can be sprayed on surfaces (wood, brick, masonry, rock, concrete, and steel) to improve their strength and increase their fire resistance. It can be applied to the underside of slabs to supplement the slab strength where conventional concreting techniques would be very difficult and sometimes dangerous. As a construction material, it can be used to construct walls, slabs (where a highly smooth surface is not needed), embankment erosion protection, and other applications where it would be difficult to use conventional concrete placement techniques. The strengths that can be achieved with shotcrete are the same as those obtained using conventional concrete in the range of 3,000 to 7,000 psi with occasionally higher strengths to 10,000 psi. Shotcrete can be placed as a mortar using only sand as aggregate, or it can contain coarse aggregate up to 3/4 in. in diameter.

129. Shotcrete is more expensive and more difficult to place than conventional concrete; however, a good portion of these costs can be recovered because the formwork costs are much less, and the cost is low for batching equipment. Batches are made in small quantities close to the placing equipment and are highly portable.

130. Procedures. During the past 50 years shotcrete has been used in the construction industry, two application approaches have been developed. Each is similar in some respect and different in others, and both have their advantages and disadvantages.

a. Wet mix process. The wet mix process is the older of the two processes and has been used in some form for about 50 years. In this process all the ingredients of the final concrete are premixed. The mixed concrete is placed into the chamber of the placement gun and conveyed to the nozzle area of the gun at a regular rate by compressed air in the gun's air hose. Additional air is added to the mixture at the nozzle to increase the velocity of the shotcrete, and it is projected onto the repair surface. In recent years this method has been used with redesigned equipment to allow the use of 3/4-in.-diam aggregate of conventional concrete mixtures to be placed by the shotcrete method.

b. Dry mix process. The newer of the two processes is the dry mix process. In this method cement and damp sand are mixed together in a small mixing container until thoroughly blended. A small 2-cu-ft concrete mixer can be used for this purpose. For smaller batches, hand mixing in a pan or other container can be accomplished. The mixing should continue until all grains of sand have been thoroughly coated with cement and then fed into the feeder chamber of the placement gun. The cement and sand are fed through the delivery hose into the mixing chamber by means of compressed air. In the mixing chamber cement and sand are combined with more water under compressed air pressure until it is thoroughly mixed. The air pressure then delivers the mortar or concrete through a nozzle and onto the repairable surface. Both methods when applied correctly will produce dense, high quality concrete. The advantages and disadvantages of each method are presented in Table 7.

131. Materials. The materials used with the shotcrete process should be the highest quality available. Where not superceded by Military Specifications, the cements should conform to American Society for Testing Materials (ASTM) Specifications: C 150-83a (ASTM 1983c) and C 595-83 (ASTM 1983d). Aggregates should conform to ASTM C 33-82 (ASTM 1982b) if normal weight material, and ASTM C 330-82a (ASTM 1982c) if lightweight material. Coarse and

Table 7

Comparison of Wet and Dry Mix Processes

Dry	Wet
Control over mixing water and consistency of mix at the nozzle.	Mixing water is controlled at the delivery equipment and can be accurately measured.
Better suited for placing mixes containing lightweight porous aggregates.	Better assurance that the mixing water is thoroughly mixed with other ingredients. This may also result in less rebound and waste.
Capable of longer hose lengths.	Less dust accompanies the gunning operation.

Note: Taken from ACI Standard 506-66.

fine aggregates under these specifications require the gradings given in Tables 8 and 9.

132. Equipment requirements. The requirements for equipment are different for the wet mix and dry mix processes. In general, mixing equipment, delivery equipment, compressed air supply, and water supply will be needed.

a. Dry mix process.

(1) Weighing and batching. The equipment to mix and batch the cement and sand must be able to thoroughly mix the two ingredients such that each sand particle is coated with cement. A rotary mixer is preferred for this task, but any container can be used that will ensure thorough mixing of sand and cement. Buckets to batch dry ingredients and weighing scales are also necessary. It is preferred that the materials be batched by weight, but they may be batched by volume if close check on the consistency of the weight in the volumes used are maintained.

(2) Delivery equipment. The delivery equipment, hose, and gun and water delivery equipment should be clean and functioning such that equipment can deliver the shotcrete in a uniform manner at a constant pressure and velocity. The discharge nozzle should be equipped with a manually operated water control so the operator can adjust the amount of water to keep the consistency of the shotcrete constant. It is important to keep the nozzle of the gun clean and free from hardened cement paste. This will ensure that the spray of mortar leaves the gun in a uniform, conical pattern. The gun should be thoroughly cleaned by flushing it with water immediately after every use to keep it in good working order.

Table 8

Grading for Coarse Aggregate

Sieve size, US standard square mesh	Percent by weight passing individual sieves		
	No. 8 to 3/8 in. size	No. 4 to 1/2 in. size	No. 4 to 3/4 in. size
1 in.	--	--	100
3/4 in.	--	100	90-100
1/2 in.	100	90-100	--
3/8 in.	85-100	40-70	20-55
No. 4	10-30	0-15	0-10
No. 8	0-10	0-5	0-5
No. 16	0-5	--	

Table 9

Grading for Fine Aggregate

Sieve size, US standard square mesh	Percent passing, by weight
3/8 in.	100
No. 4	95-100
No. 8	80-100
No. 16	50-85
No. 30	25-60
No. 50	10-30
No. 100	2-10

(3) Air supply. The compressed air supply should be capable of supplying clean, dry, oil-free air to the delivery hose. The size of the compressor should match the dimensions of the hose and nozzle. The capacities given in Table 10 are appropriate for hose and nozzle dimensions of normal shotcrete equipment.

(4) Water supply. The water supply pressure should be greater than the air supply used to drive the cement and sand to the mixing chamber to ensure that the water gets thoroughly mixed with the other ingredients.

Table 10

Compressor Capacities

Compressor capacity, cu ft/min	Hose diameter, in.	Maximum size of nozzle tip, in.	Operating air pressure available, psi
250	1	3/4	40
315	1-1/4	1	45
365	1-1/2	1-1/4	55
500	1-5/8	1-1/2	65
600	1-3/4	1-5/8	75
750	2	1-3/4	85

b. Wet mix process.

(1) Weighing and batching. The equipment and techniques used to weigh and batch the materials used in the wet process should be the same as those for the dry process. The major difference between the two techniques is that the water is added to the wet mix process mixture at the time of batching.

(2) Delivery equipment. There are two types of delivery equipment: pneumatic and positive (piston action). Both are capable of delivering the concrete to the nozzle in the proper manner. The piston equipment is a newer technique. Both types should provide the proper force to obtain the correct velocity of material exiting from the gun. The delivery hose is slightly larger than that used for the dry mix process ranging between 1-1/4 to 2-1/2 in. in diameter to accommodate the larger size aggregates that can be used with the wet mix process.

(3) Air supply. The equipment should be the same as for the dry mix process.

(4) Water supply. The equipment should be the same as for the dry mix process.

133. Personnel. A crew of five persons is needed to operate the shotcrete equipment properly. The nozzle operator should be the most experienced person on the team. This position requires the most care in applying the shotcrete to the formwork. A detailed description of this job is included in Appendix D. The delivery equipment operator makes sure that the concrete materials are delivered to the gun in a smooth, uniform manner. He is assisted with a helper to batch the materials, and he makes sure the proper delivery pressures are maintained. The nozzle operator's assistant operates

an air blow pipe. The air blow pipe deflects particles of sand which rebound from the shotcrete surface from contaminating the freshly shotcreted surface. A general assistant is needed to help lift the delivery hose as the nozzle operator moves from area to area. Other personnel not included herein could be a foreman and personnel to operate the generator for power.

134. Surface preparation. The surface preparations for accepting shotcrete are the same as for any other application of concrete. The surface should be clean, strong, and free of all foreign material. The steps that are outlined in paragraphs 86 through 117 should be followed.

Repairs Using Military Bridging Equipment

135. There are a number of items of bridging equipment in the Army inventory that are designed to transport men and equipment across streams and rivers. Some of these can also be used in an expedient situation to bridge damaged areas of piers/wharves. Both personnel and equipment bridging can be used either to span damaged areas or provide a means of traffic movement around them. The following pieces of equipment are described to suggest their potential uses in the damaged port environment.

Aluminum floating footbridge

136. Applicability. This equipment, if available, can be used to allow workmen to bypass a damaged area on a pier/wharf while repair is being conducted. This floating walkway can also be used as a support for hoses or other conveyances of liquids needed on a pier to keep portions of the wharf productive while repairs are made to another section.

137. Use. This bridge is a personnel type bridge composed of prefabricated sections suitable for field assembly. It consists of sections of aluminum treadway with a path width of 1 ft 8-1/2 in. which are linked together and secured to aluminum pontoons 14 ft long, 2 ft wide, and 1 ft 2-1/2 in. deep. Figure 47 shows the launching of a completed footbridge.

138. Crew. A complete set of aluminum footbridge components provides a floating walkway 472 ft long which can be assembled by one platoon of men in less than 4 hr. It can be land transported on two 2-1/2 ton cargo trucks equipped with pole-type trailers, and the footbridge components can also be transported aboard a C-130 aircraft. It can be assembled in parallel tracks to provide a floating bridge 100 ft long for light vehicles such as jeep and

Figure 47. Completely assembled footbridge

trailer. One such bridge set is issued to each Corps or Army float bridge company.

Light tactical floating bridge

139. Applicability. This equipment, if available, can be used to provide a floating roadway for light truck traffic (Class 9 loading in 3 ft/sec current) to bypass a damaged section of pier/wharf while repair is being conducted. This floating bridge can also be used as a supporting structure for hoses or other conveyances of liquids needed on a pier to keep portions of the wharf productive while repairs are made to another section.

140. Use. The floating tactical bridge consists of equipment supplied in raft sets. Each raft set provides the components to construct one four-pontoon reinforced raft or a 44-ft long normal bridge. The raft sets consist of eight half pontoons, eight deck panels, eight filler panels, and end ramps. This is enough to construct a 44 ft long by 9 ft wide bridge. Figure 48 shows a partially completed four-pontoon raft bridge.

141. Crew. Bridging assembled from light tactical raft sets takes varying amounts of time to assemble depending on weather conditions, sea conditions, and length of bridge. Assembly could range from 1 to 6 or 7 hr depending on the applicable factors. A normal rate of assembly for experienced troops at a prepared site is about 10 ft/min for normal assembly. One raft set can be land transported on two 2-1/2 ton cargo trucks equipped with pole-type trailers, and it can also be transported aboard a C-130 aircraft as well as by helicopter. The light tactical raft set is issued on the

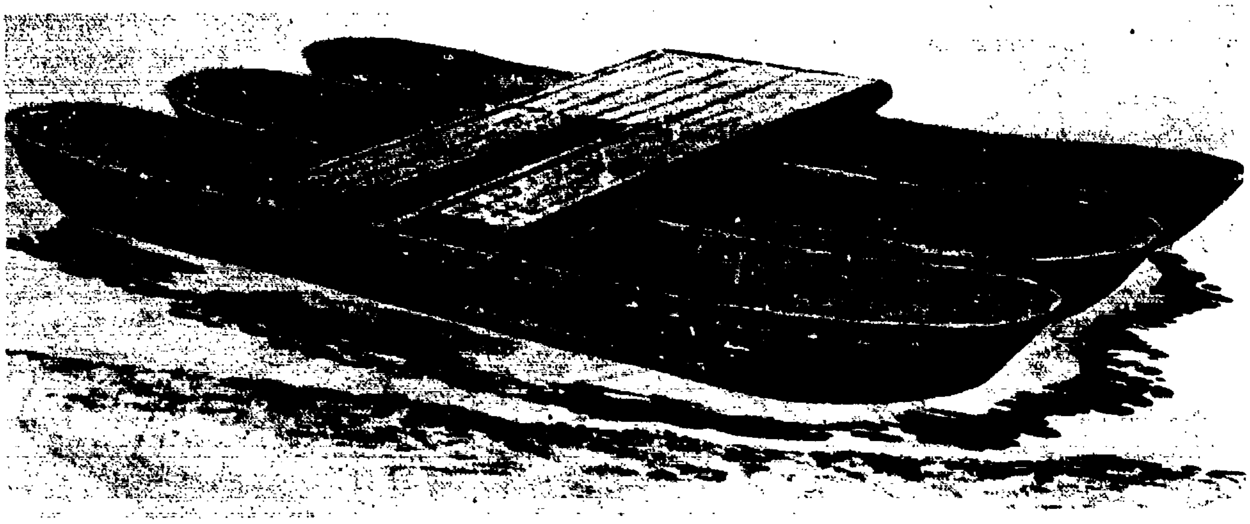


Figure 48. Partially assembled light-tactical floating bridge

basis of six per engineer float bridge company, and two per division engineer battalion.

M4T6 floating aluminum bridge

142. Applicability. This bridge set can be used in either a floating mode or in a modified condition to span a damaged section of pier/wharf. It has a capacity of carrying heavy loads of military vehicles up to 60 tons in calm waters with currents less than 3 ft/sec and can be slightly overloaded if necessary. In the floating mode, the bridge can be used to divert truck and tracked vehicle traffic around a damaged section of pier/wharf. The modified bridge can be used to span and convey these vehicles over a damaged section of pier/wharf when placed on the deck.

143. Use. The M4T6 is supplied in bridge sets. Each set consists of adequate floats, substructure, bearing beams and deck structure to construct a roadway approximately 142 ft long and 13 ft wide. Each division engineer battalion is issued four sets of the M4T6, and there are five additional sets issued to the Corps/Army float bridge company. The basic construction of the bridge consists of balk beams or the deck members which rest on balk connecting stiffeners which attach to saddle beams (Figure 49). These components assemble to form the structural deck and substructure. If the sets are used as floating bridge structures, the above assembly is then connected to the raft portions of the set with saddle adapters. If the bridge is to span an open space on a pier/wharf, the deck and substructure are modified to fit the

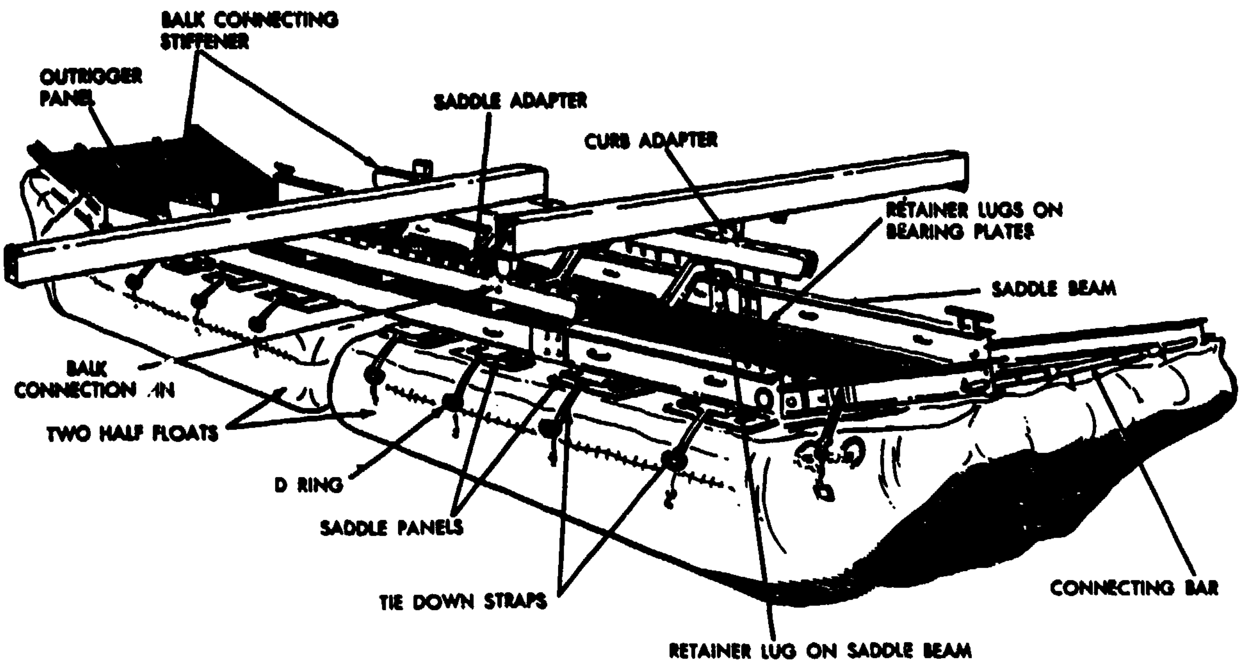


Figure 49. Float and substructure of M4T6 floating aluminum bridge

foundation constraints. Details of construction are described in TM 5-210 (Headquarters, Department of the Army 1970).

144. Crew. The crew necessary to assemble the M4T6 sets varies with the length of bridge being constructed including other factors such as time of day, weather conditions, combat conditions, water currents, etc. In ideal conditions, a 250-ft bridge can be constructed by one construction company supported by two to three float-bridge platoons in approximately 6 hr. The construction equipment required includes three air compressors, three cranes, and four power boats. This bridge is hand-erectable and air-transportable as opposed to the Class 60 bridge. The M4T6 bridge is capable of carrying division type loads. It is normally transported on land by the six by six military bridge truck, but it can be transported on any standard military cargo truck having the capacity of 2-1/2 ton or more. Each set requires five trucks to carry the component parts: normal bay, offset bay, trestle, anchorage, and tool and rigging trucks. Details of these are described in TM 5-210 (Headquarters, Department of the Army 1970).

145. Remarks. The M4T6 and the M4 bridge sets are the items of equipment that will most likely be available for use to temporarily span damaged areas. This equipment has been in the Army inventory for many years and has been succeeded by more sophisticated equipment for assault bridging in the

forward areas. They are therefore more likely to be available for port repair tasks.

M4 floating bridge

146. Applicability. The M4 floating bridge is useable anywhere the M4T6 would be, and both serve the same purpose of floating or fixed bridge sections.

147. Use. The M4 bridge has the same capacity (60 ton) as the M4T6 under normal conditions, but can be loaded to approximately 100 ton when reinforced. It is a wider bridge section which provides 13 ft 10 in. between curbs. The M4 bridge is not available as organic equipment to any unit and is considered a standard B item. It is issued in the same type of sets as the M4T6 bridge. The M4 differs from the M4T6 in that it is equipped with aluminum pontoons rather than inflatable rafts as shown in Figure 50.

Figure 50. Completed section of M4 floating bridge

148. Crew. Since the M4 bridge sets are not issued as standard issue, there is no standard method of assembling them. However, TM 5-210 (Headquarters, Department of the Army 1970) gives a recommended method of assembly and the number of personnel required. The bridge can be assembled with a crew consisting of 176 enlisted men and 13 noncommissioned officers. The crew is assisted with two cranes and three bridge erection boats and other small tools integral to bridge construction sets. The M4 bridge does not come with any particular transportation recommendations; therefore, the use of any available military transportation is recommended.

Class 60 floating bridge

149. Applicability. The Class 60 floating bridge is designed to move the Army's Class 65 tracked vehicles and other heavy equipment not appropriate for other bridging situations. It can be used in a floating condition, or if

modified can span fixed areas such as damaged pier/wharf decks. It is similar to the M4T6 bridge with the exception that it is composed of steel grid panels rather than aluminum.

150. Use. The Class 60 bridge is organized and issued in sets similar to the M4T6. One standard bridge set contains components for assembly of one floating bridge 135 ft long and 13 ft 6 in. wide. Four sets are issued to each division engineer battalion, and five sets are issued to each Army/Corps float bridge company.

151. Properties. Unlike the aluminum M4T6 bridge, the Class 60 bridge is made of steel and is therefore heavier. This limits its transportation primarily to land based methods and it is not air deployable by helicopter. It has inflatable pontoons similar to the M4T6 bridge and can be deployed by 6 by 6 military bridge trucks. Usually, one bridge truck will carry a complete 15 ft bay of bridge, requiring nine bridge trucks to carry all the sections to complete one set. Additional trucks are needed to carry the trestle and anchorage load, the ramp load, and the erection equipment load.

152. Crew. The total number of personnel required for construction at one site is approximately 92. If there is more than one construction site, an additional 60 personnel are required at each site to conduct operations concurrently. The time for personnel to assemble the Class 60 bridge will vary, but under good weather conditions and daylight operations, 90 ft length of bridge can be erected in the first hour and 120 ft each hour thereafter.

Fixed span bridging from floating bridge equipment

153. Light tactical raft. Fixed span bridges up to 38 ft can be assembled from components of the light tactical raft. Deck panels, ramp panels, and deck filler panels can be assembled and launched across a gap to form a deck that will support the loads given in Table 11.

154. M4 and M4T6 spans. When these aluminum bridge sets are used to span a fixed gap, spans can range from 15 to 45 ft without intermediate support, and can be expanded even farther using trestles as shown in Figure 51. Since these bridge sections are supported on fixed foundations they can support loads greater than when used as a floating bridge. Table 12 gives the relationships of load, span, and road width. Trestle supports will need to coincide with pile caps when using multiple span sections. Span lengths will have to be chosen to fit the existing substructure.

Table 11

Load Classifications for Light Tactical Raft Spans

Clear Span ft	Load Class Normal	Load Class Caution	Load Class Risk
20	21/17	25/19	32/23
22	18/15	20/17	23/19
24	16/13	18/15	20/17
26	14/12	16/14	18/16
28	12/11	14/13	16/15
30	11/10	12/12	16/14
32	10/9	11/11	15/13
34	9/8	10/10	13/12
36	8/7	10/9	11/12
38	7/7	9/9	10/11

Note: Wheeled load class/tracked load class.

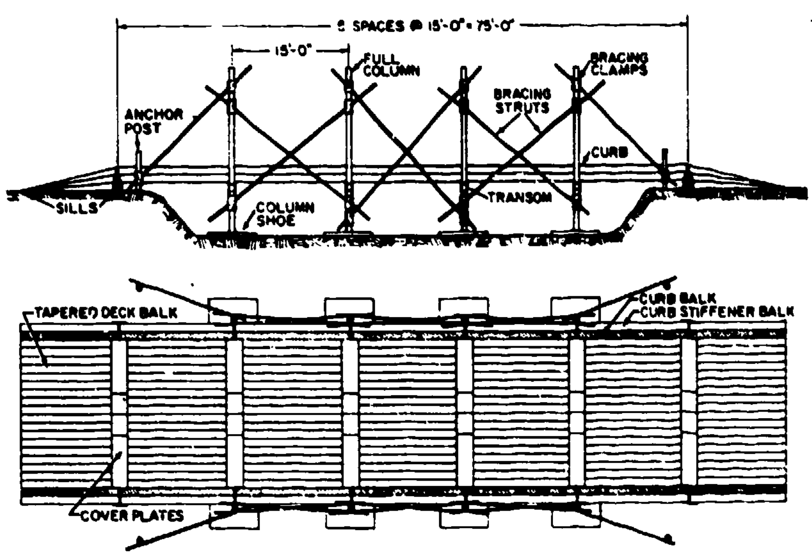


Figure 51. Multispan fixed bridge on 50-ton trestle

Table 12

Load Classifications for M4 and M4T6 Fixed Spans

Clear Span ft	Deck Width No. Balk	Road Width No. Balk	Load Class Normal	Caution	Risk
15	22	18	120/100	120/100	120/100
23-1/3	22	18	120/100	120/100	120/100
30	22	18	85/65	100/80	110/90
	22	16	90/70	100/80	110/90
	24	18	90/70	105/85	115/95
38-1/3	22	18	45/35	70/51	78/57
	22	16	50/40	70/51	78/57
	24	18	55/45	75/55	85/62
	26	18	65/50	82/60	90/67
45	20	16	24/25	40/35	47/40
	22	18	24/25	46/40	54/45
	22	16	30/30	46/40	54/45
	24	18	30/30	51/43	60/49
	24	16	40/35	51/43	60/49
	26	18	40/35	56/46	66/53
	26	16	45/40	56/46	66/53

Note: Wheeled loaded class/tracked load class.

155. Class 60 fixed spans. Clear spans up to 60 ft can be bridged using the Class 60 sections. With clear spans as short as 24 ft, loads up to Class 120 can be accommodated. Spans that are longer than 60 ft can be achieved using intermediate supports with trestles. The details of assembly, loading and transportation for this expedient type of bridge are described in TM 5-210 (Headquarters, Department of the Army 1970).

Fixed and mobile bridging

156. Fixed span and mobile counterparts of the floating bridge equipment include the Bailey Bridge sets, the Medium Girder Bridge, and the Armored Vehicle Launched Bridge. These bridging sets can be used to either permanently or temporarily span a damaged pier/wharf in a port environment. A brief description of each is given in the following paragraphs.

157. <u>The Bailey Bridge, Type M2.</u> The Bailey Bridge is a versatile, easily assembled, through-type truss bridge which can be constructed in a matter of hours, carry heavy military loads, and span up to 210 ft without intermediate support. It is versatile because panels are fabricated to measure 5 ft- 1 in. wide by 10 ft long, and they can be assembled in a large variety of lengths, heights, and configurations. The bridge provides a roadway that is 12-1/2 ft wide from curb to curb and can support as much as a 90 class loading under certain normal conditions. The trusses which flank the roadway can be constructed in one, two, or three truss-side-by-side combinations, as well as combinations one, two, or three stories high to impart different span and loading capabilities to the bridge. The roadway is normally supported by transoms set between the lower cord of the truss such that the truss is above the roadway as shown in Figure 52; however, if the situation warrants it, the

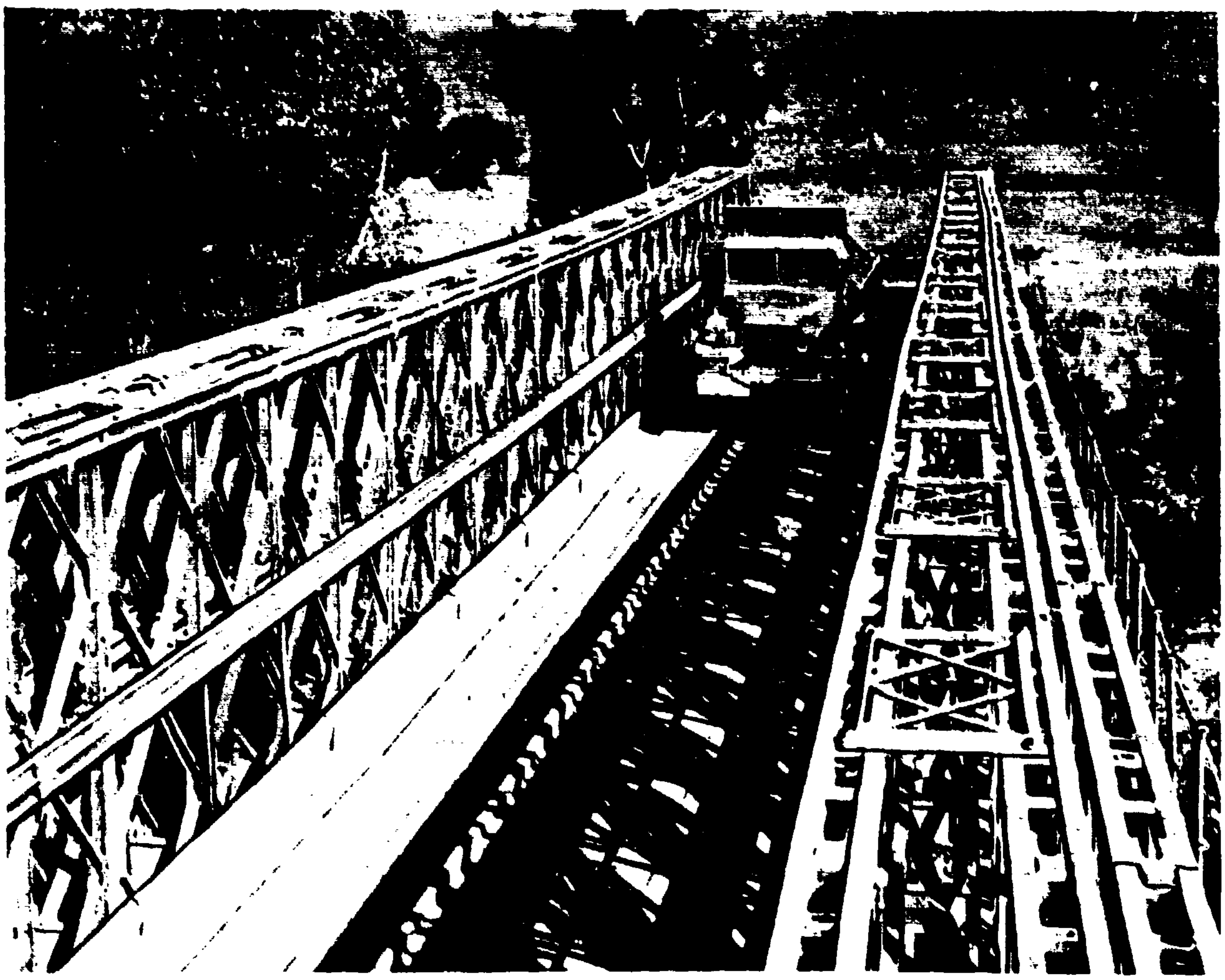


Figure 52. Example of a triple-truss double-story Bailey Bridge

roadway can be constructed on the top cord of the trusses with the supporting structure of the bridge below the roadway. This can be seen in Figure 53. As conceived for use in the port environment, the Bailey Bridge would be similar to the artist's concept shown in Figure 54.

158. The basic components of the M2 Bailey Bridge sets are not issued as normal stock; however, they are available at Army depots. A standard set contains enough parts to construct two 80 ft double-single M2 bridges, or one 130 ft double-double bridge. The standard set can be transported on eight 5-ton dump trucks and two 2-1/2 ton trailers attached to two of the trucks. The engineer company (panel bridge) normally transports the sets with assistance of two platoons. Table 13 gives the number of trucks and trailers required for various spans and configurations.

159. The number of men required to assemble a Bailey Bridge depends upon its configuration and its span. The crews are generally organized into unloading parties and assembly parties. Each unloading party consists of one noncommissioned officer (NCO) and eight enlisted men (EM's). Table 14 gives the necessary unloading parties for various spans. The assembly parties consist of varying numbers of NCO's and EM's depending upon the composition of the bridge and whether or not the use of cranes are available. The breakdown is given in Table 15. The time to assemble a bridge depends upon the experience of the crew, the weather and visibility, the setup of the site, and a number of other considerations. If all conditons are favorable, the times given in Table 16 can be assumed.

160. The use of cable reinforcement in conjunction with a Bailey Bridge will increase its load-carrying capacity. The cable reinforcement set for the M2 set increases to Class 60 wheel and track, the classification of triple-single bridges for span lengths from 100 ft to 170 ft. For a span of 180 ft the Class is 50 wheel and 60 track. The additional cable reinforcement set significantly reduces assembly time and equipment necessary for Class 60 traffic to cross spans between 100 and 180 ft. The cables are attached to the ends of the span and are strung over vertical spacers mounted on the underside of the truss at predetermined locations. The cables are tensioned causing the bridge to deflect upward. When a vehicle crosses the bridge, the bridge deflects downward transferring a portion of the load to the cables. Figure 55 shows a triple-truss single-story bridge augmented with the cable reinforcement set.

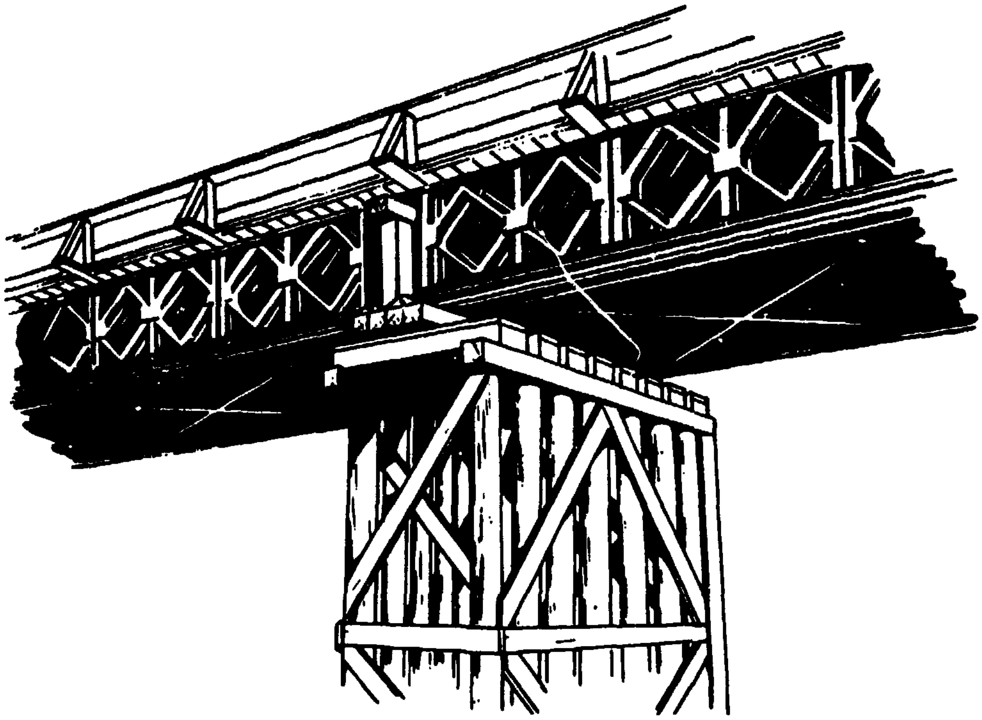


Figure 53. Roadway on top chord of single-story Bailey Bridge.

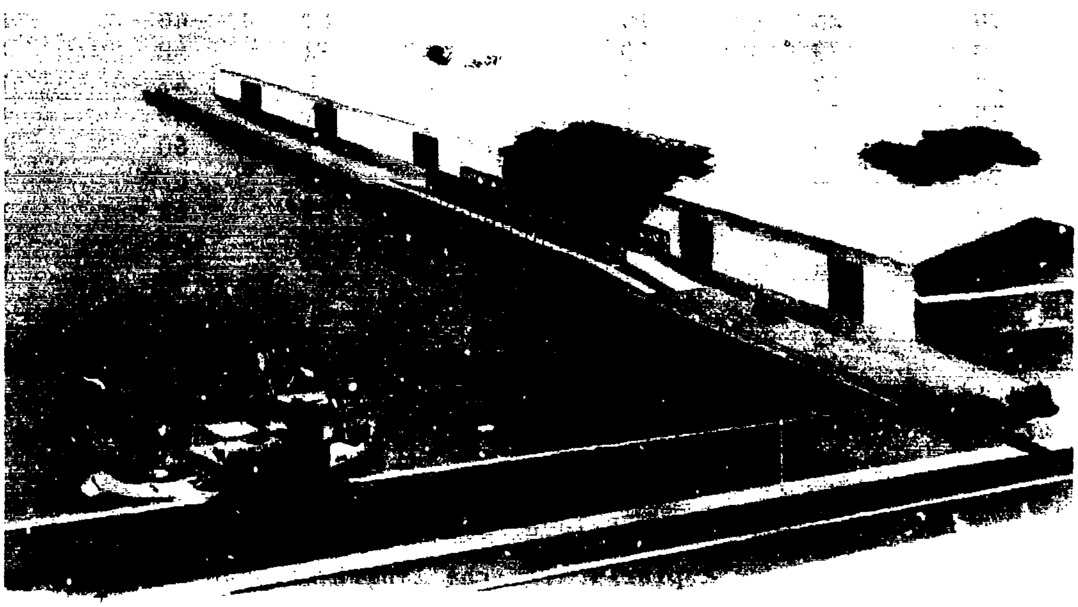


Figure 54. Artist's concept of Bailey Bridge use for port repair

Table 13

Trucks and Trailers Required for Various Spans and Assemblies of Type M2 Bailey Bridge

		Total Vehicles				Total Vehicles	
Span	Type	5-ton trucks	2-1/2-ton trailers	Span	Type	5-ton trucks	2 1/2-ton trailers
30	SS	9	3	140	DS	20	8
40	SS	9	3		TS	24	8
50	SS	12	3		DD	28	8
	DS	14	5		TD	34	8
60	SS	11	4		DT	33	11
	DS	13	4	150	TS	23	8
70	SS	11	4		DD	30	8
	DS	13	5		TD	36	8
80	SS	12	5		DT	38	8
	DS	14	5	160	TS	25	8
	TS	18	7		DD	31	8
90	SS	14	6		TD	39	8
	DS	16	6		DT	39	8
	TS	19	6		TT	49	13
100	SS	15	5	170	DD	36	10
	DS	17	5		TD	45	10
	TS	19	6		DT	44	10
	DD	23	9		TT	56	15
110	DS	17	7	180	DD	36	10
	TS	20	7		TD	47	10
	DD	24	10		DT	46	10
	TD	29	10		TT	58	10
120	DS	18	7	190	TD	47	11
	TS	21	7		DT	49	11
	DD	25	7		TT	60	11
	TD	30	10	200	DT	52	11
130	DS	19	7		TT	62	11
	TS	22	7	210	DT	54	11
	DD	26	7		TT	68	11
	TD	34	7				
	DT	33	11				

Note: S = Single, D = Double, T = Triple; Example: DT = Double-truss Triple-story.

161. When using the Bailey Bridge in conjunction with temporarily repairing a damaged area of a pier/wharf, the length of span should be as short as possible to avoid the dead weight of the bridge adding to sag in the span. Generally, intermediate piers should be used to avoid assembly of Class 50 continuous spans longer than 150 ft or Class 75 continuous spans longer than 120 ft. Bridges supported by piers may be either broken (at each

Table 14

Number of Unloading Parties

Type	Span, ft	Parties
SS	30-60	3
	70-90	4
DS	50-80	4
	90-120	5
TS	70-120	5
	130-140	6
DD	90-150	6
	160-170	7
TD	110-120	6
	130-180	7
	190-200	8
DT	130-170	7
	180-200	8
TT	150-200	8

Table 15

Organization of Assembly Parties

Type	NCO's-EM's
SS	4-39
DS	4-42
TS	5-58
DD	5-66
TD	6-92
DT	7-122
TT	7-148
DT*	7-97
TT*	7-103

* Denotes use of one crane and a three-man crew.

Table 16

Estimated Times For Assembly

Span	Type	Time, Hr	Span	Type	Time, Hr
40	SS	1-1/2	160	TS	5
60	SS	1-3/4		DD	6-1/4
	DS	2		TD	8-1/2
80	SS	2		DT	13-1/4
	DS	2-1/2		TT	19
	TS	3		DT*	11-3/4
100	SS	2-1/4		TT*	16-1/4
	DS	3	180	DD	7
	TS	3-1/2		TD	9-1/2
	DD	4-1/4		DT	14-3/4
120	DS	3-1/2		TT	21-1/4
	TS	4		DT*	13-1/4
	DD	5		TT*	18-1/4
	TD	6-3/4	200	DT	16-1/4
140	DS	3-3/4		TT	24
	TS	4-1/2		DT*	14-1/2
	DD	5-3/4		TT*	20-1/2
	TD	7-1/2			
	DT	11-3/4			
	DT*	10-1/2			

* Denotes use of one crane and a three-man crew.

pier) into separate spans or continuous for their entire length. Any type of supporting substructure, whether wood, concrete, or steel may be used to support the Bailey Bridge provided that the structure is capable of supporting the loads which will be applied to it. It can be supported on several of its own trusses configured as a column if there are no available supports for the span. FM 5-277 (Headquarters, Department of the Army 1986a) contains necessary details to help configure the Bailey Bridge.

162. If the bridge is to be supported by pile foundations, it is desirable that all the pile tops which support the bridge should be the same elevation, but a small change of elevation between spans can be tolerated. If timber trestle or pile bents are used as intermediate piers, construct the top of the bent as shown in Figures 56 through 60. If end posts are not used, reinforce the cap sill with a steel bearing plate under each line of trusses. The plate should be sized to match the number of trusses which will be attached together. On single bents, use corbels with knee braces to provide a jacking platform for light bridges (Figure 57). If double bents are used with

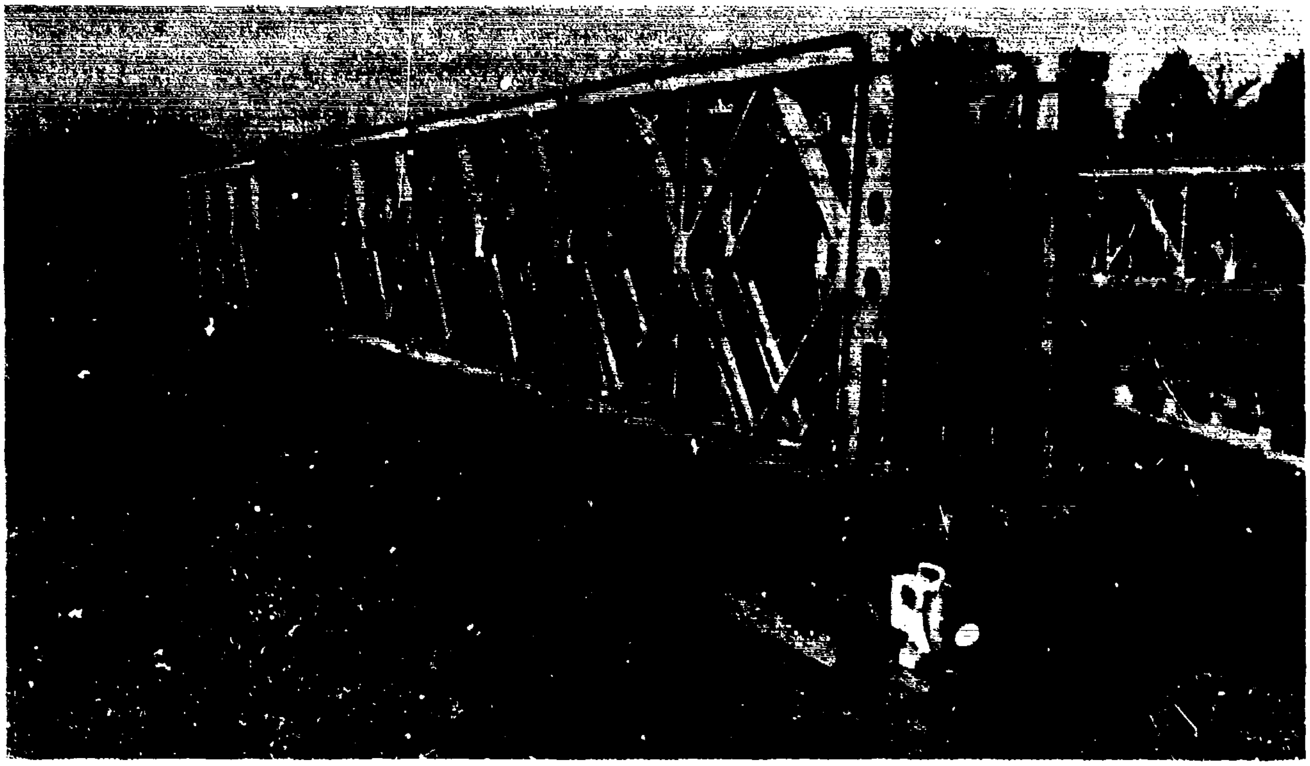


Figure 55. Bailey Bridge with cable reinforcement set

end posts and standard bearings, lay timbers across caps to provide platforms for seating bearings (Figures 59 and 60) and group timbers together under each line of trusses.

163. Bailey Bridges can be used as expedient bridging for the assembly of railway bridges, or to support gantry crane tracks on piers/wharves. However, they should be used only under special conditions since there is a great amount of deflection under such heavy loads. Spans longer than 70 ft require special configurations (quadruple-truss double-story) to support the weight. Table 17 gives the loadings for locomotives. Greater detail for railroad bridge used as well as for any topic mentioned in this section is discussed in FM 5-277 (Headquarters, Department of the Army 1986a).

164. Medium Girder Bridge. The Medium Girder Bridge (MGB) is a lightweight alternative to the Bailey Bridge which can carry heavy loads up to Class 60. The bridge is quicker to erect and requires only bridge unit personnel for erection. The main advantages of the MGB are: it is quickly and easily assembled by hand, no site preparations are required, it is easily transported by road or air as palletized loads, it is light and sturdy, little

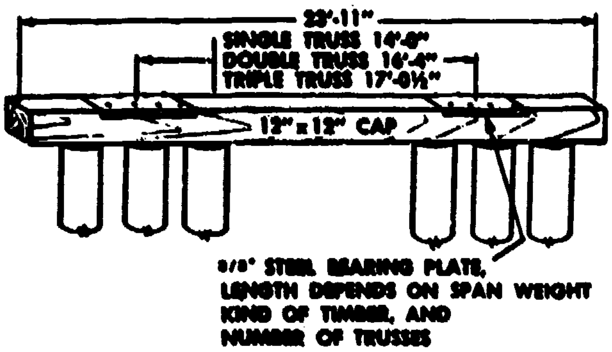


Figure 56. Construction of pile caps for intermediate supports under panel bridges without end posts or bearings. Cap of single timber or pile bent. Steel bearing plates provide seating for each line of trusses.

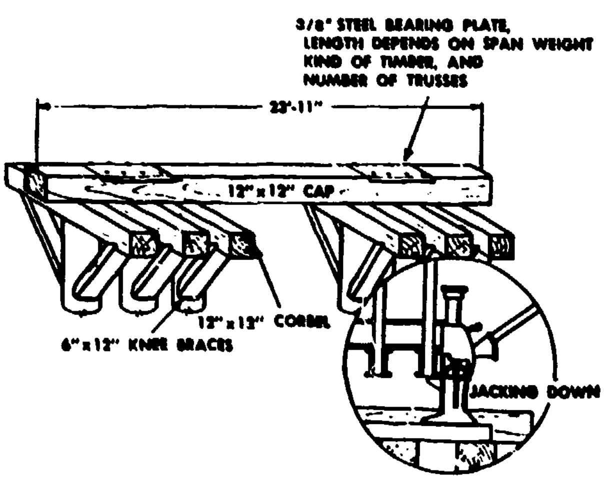


Figure 57. Construction of pile caps for intermediate supports under panel bridges without end posts or bearings. Corbels and knee bracing added to provide a jacking platform for light bridges.

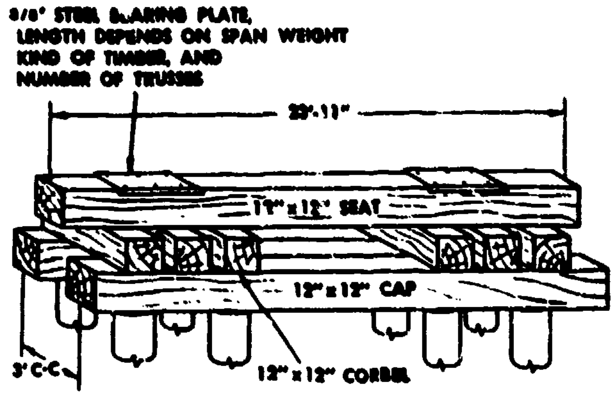


Figure 58. Construction of pile caps for intermediate supports under panel bridges without end posts or bearings. Cap for double trestle or pile bent.

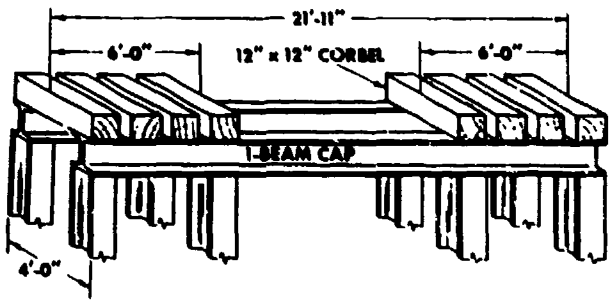


Figure 59. Construction of pile caps to provide seating for bearings at adjacent ends of independent spans-steel piles and caps.

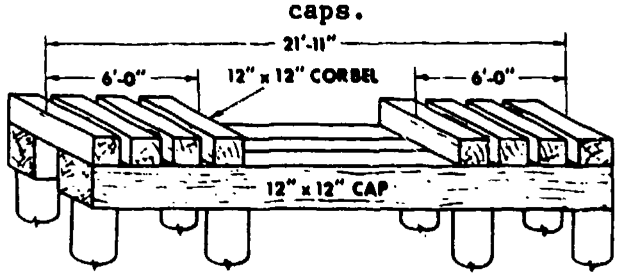


Figure 60. Construction of pile caps to provide seating for bearings at adjacent ends of independent spans-wood piles or trestle and cap.

Table 17

Maximum Live-Load Moments and Live-Load Shear Caused by Cooper's E-45 Loadings

	Standard Cooper's E-45 loading (Two Locomotives)		Modified Cooper's E-45 loading (One Locomotive)	
Span, ft	Maximum Shear ton	Maximum Moment ft-ton	Maximum Shear ton	Maximum Moment ft-ton
10	34	63	34	63
20	56	232	56	232
30	71	462	71	462
40	85	739	85	739
50	98	1,060	98	1,060
60	110	1,460	110	1,446
70	124	1,920	122	1,845
80	139	2,440	134	2,303
90	154	3,000	145	2,791
100	169	3,630	158	3,312

training is required, it has a multispan capability, a portable pier set is available, it has loading capacities of up to Class 60, and little or no maintenance is required.

165. The MGB consists of girders 6 ft long and 1 ft 9 in. deep that fit together using pins and provides a deck with a roadway which is 13 ft 2 in. wide from panels that are attached between the girders. The single story bridge (SS) is composed of these girders linked together. This is appropriate for carrying light loads or heavy loads for short spans. The SS provides Class 60 bridging up to 32 ft, decreasing to Class 16 at 74 ft, which is its maximum length. When the SS is augmented with triangular truss pieces that attach beneath the 6 ft girders, it becomes a double story bridge (DS). The DS provides Class 60 bridging up to 103 ft, decreasing to Class 16 at 163 ft, which is its maximum length. All components are made of a weldable alloy of aluminum, zinc, and magnesium which provides for lightweight components of the bridge. There are seven different components that constitute the bridge, all but two of them weigh less than 440 lb such that four individuals can handle.

The other two weigh less than 570 lb and require a six-man team for lifting. Because of its light weight, it can be assembled by hand without the need of a crane.

166. The MGB sets can be assembled by the existing crew of a MGB company in less than 2 hr depending on the span and conditions given in Table 18. Because of its simplicity, the crew does not need any specialized training for assemblage. These times may be achieved on typical sites by personnel who have some experience in building similar bridges. These building times are based on both day and night trials. Times begin with the arrival of the first load of components and end when the bridge is ready for operation. The work parties that are required to construct the MGB are given in Table 19.

167. The MGB components are palletized and adapt to the bed of the M796 trailer for transportation. The pallets are designed to remain with the bridging when it arrives at the construction site and can be unloaded by securing the pallet to a fixed object and driving the trailer from underneath

Table 18

Bridge Building Times

	Bridge Span Building Times, min			
Conditions	32 ft Class 60 SS	50 ft Class 30 SS	74 ft Class 16 SS	103 ft Class 60 DS
Actual Time/Day	15	42	47	90
Actual Time/Night	35	57	62	115
Planned Time/Day	30	45	60	120
Planned Time/Night	45	60	75	145

Table 19

Work Parties for the MGB

Span	Party
SS up to 32 ft	1 NCO + 8 men
SS 38 to 50 ft	1 NCO + 16 men
SS 50 to 74 ft	1 NCO + 16 men
All DS spans	1 NCO + 24 men

the pallet. The end of the pallet is cushioned to break the fall. There are four bridge sets allocated to each Corps engineer MGB company.

168. Two additional features make this bridge more versatile. The reinforcement kit, which is similar to the reinforcement kit for the Bailey Bridge, will extend the capacity of the bridge to Class 60 at 162 ft. An additional party of eight men assemble this component while the other components are being fitted together. The portable pier set, which is the other feature, provides the MGB with legs or columns up to 40 ft in length for use where it is desirable to reduce the clear span of the bridge. The portable pier set is transported on a standard pallet and the heaviest component weighs 900 lb. An artist's concept of the MGB spanning a damaged pier is shown in Figure 61.

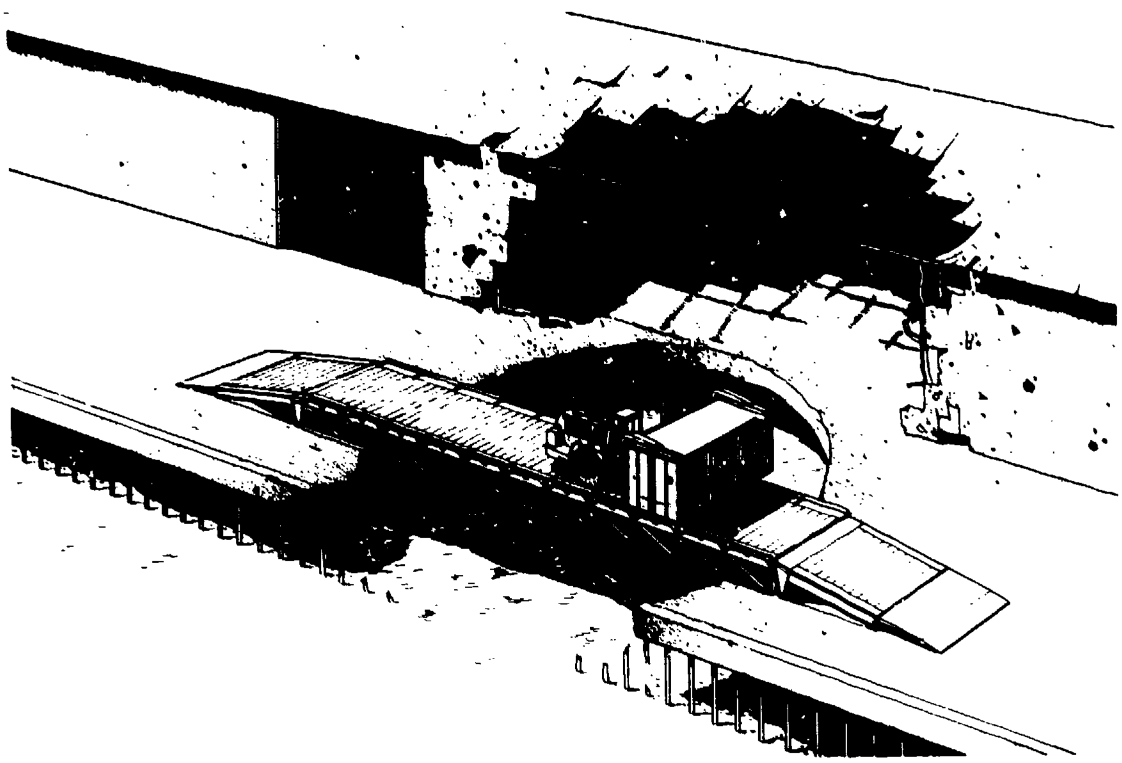


Figure 61. Artist's concept of a MGB spanning a war-damaged pier

169. <u>Armored Vehicle Launched Bridge.</u> The Armored Vehicle Launched Bridge (AVLB) is a mobile assault bridge designed for use in the forward eschelons to cross ditches, rivers, and other depression obstacles. If available, it provides a temporary repair of piers/wharves by providing an immediate bridge over a damaged section which is less than 60 ft long.

170. The AVLB is an aluminum scissors bridge that adapts on the chassis of a converted M48 or M60 main battle tank (MBT). The bridge and chassis weigh approximately 57 ton. When deployed, a bridge is provided that is 60 ft long and 12-1/2 ft wide. It has a capacity for Class 60 vehicles. The bridge is carried in the folded position and is hydraulically launched over the front of the vehicle. The bridge is launched as follows: the AVLB is driven up to the obstacle and halted; the bridge is hydraulically raised into the vertical position, unfolded and lowered into place; and the launcher is detached.

171. The bridge can be deployed in approximately 2 to 5 min, and recovery time can range from 10 min. to 1 hr depending on the condition of the ground on which it was launched. There are four launchers and six bridges issued to each Engineer Battalion of Armor, Infantry, or Mechanized Infantry Divisions and two launchers and bridges in each Tank Battalion. Figure 62 shows an artist's concept of the AVLB being launched on the deck of a damaged pier.

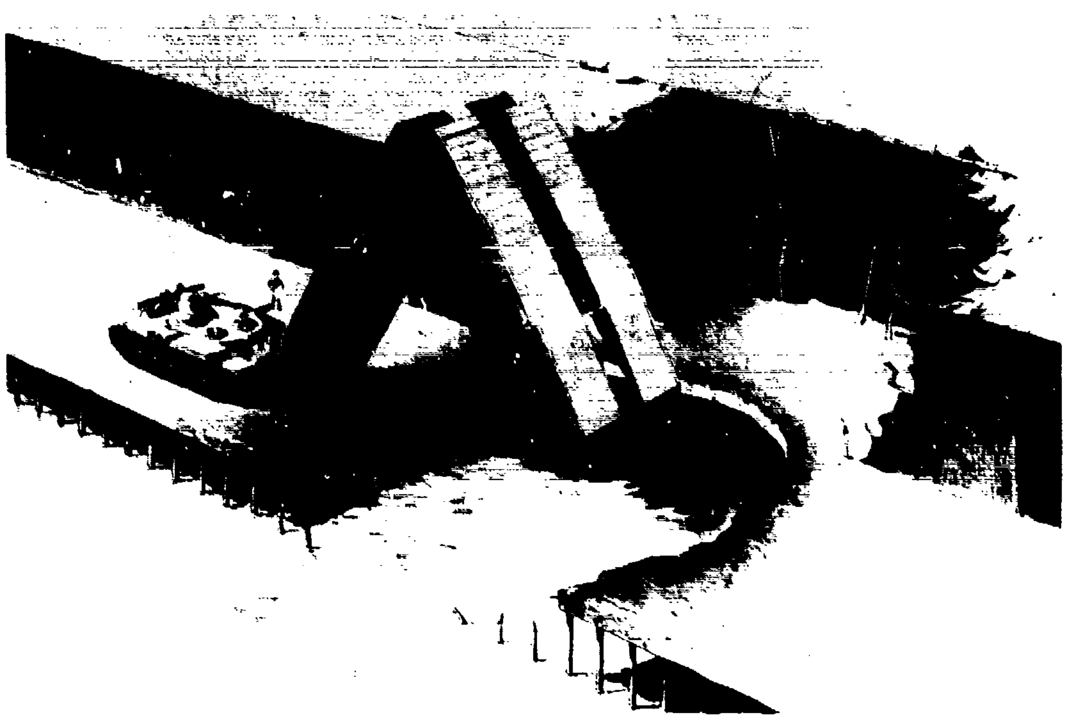


Figure 62. Artist's concept of AVLB in a war-damaged port environment

172. Due to the versatility of this bridge and its obvious utility in forward eschelons, it is unlikely that this bridge would be available for pier

repair efforts in the Echelons Above Corps. It is included as a possibility for temporary pier repair measures in the event that it is available. Since its deployment weight is approximately 57 ton and since it must be launched from a stable location, the capacity of any pier over which the launcher must travel should be known in advance to prevent loss of equipment or additional damage to pier structures.

173. DeLong Barges and other Spud Barges. The self-elevating spud barge commonly known as the DeLong barge consists of a barge unit, a number of pneumatic jacks, control panels, caissons, and miscellaneous equipment. The barge is manufactured by the DeLong Corporation which provide two sizes of barges being used in the military inventory. The "A" DeLong barge is 300 ft long, 80 ft wide, and 13 ft deep. It is supported by ten 6-ft-diam caissons and ten 500-ton-capacity jacks. The "B" DeLong barge is 150 ft long, 60 ft wide, and 10 ft deep. It is supported by six 6-ft-diam caissons and six 500-ton-capacity jacks.

174. The DeLong barge is a welded steel honeycomb-like structure consisting of plates and stiffeners. It is designed to support deck loads of up to 500 psf. In its normal operations mode, 4- by 12-in. planks are laid over and secured to the steel deck plate to provide a nonskid working surface. Detailed information such as description, site selection, preparation of barge for erection, erection procedures, preparation for use and removal are outlined in TB 5-360-1 (Headquarters, Department of the Army 1968).

175. Several of the "A" DeLong barges could be placed together end to end to provide a temporary pier that could be used to offload cargo until the conventional pier/wharf had been repaired. However, due to deck design, the barge would not be able to handle the heaviest container port loads without modifications.

176. Major limitations of DeLong barges to be used in container port construction include:

a. Erection time. The best information available indicates that each barge requires from 1 to 8 days for positioning and erection. The time depends on the characteristics of terrain and crew efficiency.

b. Structural integrity. DeLong barges appear to be designed for a 500- to 600-psf live load. Military container ports are designed for 1,000-psf live loads. DeLong barges, without modification, are therefore incapable of meeting this container port structural requirement. A 12-in.-thick timber

deck is included in TM 5-302 (Headquarters, Department of the Army 1986b) to strengthen the DeLong barge for container loadings.

c. Container handling capability. DeLong barges which are modified to accommodate container loads are used to construct a temporary installation. The barge is jacked to the desired elevation, the grippers within the jacks are inflated to 350 psi, and the shutoff locks and isolation valves are closed. The height of the caissons above the deck severely affect the off-loading performance of most container handling cranes which must boom up and down to keep from striking individual caissons.

d. Relocatability. Temporary installations fabricated with DeLong barges are not easily relocated. Semipermanent installations which have caissons driven to refusal, cut off, and then welded to the barges are much more difficult, and seldom is it feasible to even attempt to relocate them.

e. Foundation limitations. Suitable foundations are usually limited to consolidated clays and nonplastic materials.

177. There are a number of other spud barges in the Army inventory which may be used as temporary piers/wharves. Table 20 gives unit data. These barges are prefabricated steel units supported by a number of spuds or caissons to elevate the barge above the water level. They are similar to the DeLong barges; however, they incorporate 100-ton jacks to lift the barge as compared to 500 ton jacks for the DeLong barge.

Table 20

Barge Unit Data

National Stock Number (NSN)	Size
1945-00-575-0570	56 ft × 45 ft × 6 ft
1945-00-591-4314	250 ft × 60 ft × 10 ft
1945-00-573-5227	300 ft × 90 ft × 13 ft
1945-00-592-4313	427 ft × 90 ft × 15 ft

178. The barges have a barge-like design to give them stability and to enable them to be floated to their destination. The barge unit is of steel construction with a honeycomb-like interior of transverse and longitudinal stiffening bulkheads designed to support heavy deck loads. The bulkheads are

also arranged to provide compartments to receive wells 6 ft in diameter into which the caissons or spuds are inserted. A 3-in.-thick by 12-in.-wide wood decking is laid over and secured to the steel deck plate to provide a nonskid deck surface.

179. Erection will normally be assigned to the Engineer Port Construction Company. The construction time for the 300-ft-long barge under ideal conditions with a crew trained for a minimum of 10 days is 24 hr using two 10-hr shifts and a 4-hr preoperation maintenance period.

180. Detailed information such as description, site selection, preparation of barge for erection, erection procedures, preparation for use and removal, etc. are outlined in TM 5-360 (Headquarters, Department of the Army 1964).

PART IV: ALUMINUM EXTRUSIONS FOR PORT REPAIR

Introduction

181. Methods and materials to expeditiously repair war-damaged port facilities are required to restore port areas for the transfer of military supplies from support ships to shore facilities and inland. A port vulnerability study which is presented in Part II of this report estimates physical structural damage to a civilian port subjected to an aerial general purpose bomb attack. Based on the study, a bomb hole of 8.4 ft in diameter can be expected from a 500-lb general purpose bomb detonating on a 12-in.-thick concrete pier/wharf deck. Expedient repair materials are needed to repair decking and thus maintain operation of container handling equipment.

182. Traffic tests were conducted at WES on truss web aluminum extrusions to field evaluate their span capability for repair of bomb-damaged pier/wharf decks. The extrusions were hinged together and placed over 4-, 6-, and 8.4-ft-diam simulated wharf deck holes. Extrusion covered holes were trafficked with a 43,900-lb single-wheel load with a tire inflated to 125 psi and also trafficked with a 16,450-lb dual-wheel load with tires inflated to 95 psi. These loadings and tire pressures represented specific port equipment expected to operate on pier decks. The extrusions were evaluated to determine the adequacy of their repair and the resulting performance in spanning a bomb hole without structural failure.

Extrusion Description

183. Twelve 24-ft-long truss web aluminum extrusions were extruded and furnished by Taber Metals, Inc., Russellville, AR. In the early 1970's, Dow Chemical Company, Midland, MI, designed and provided WES several test quantities of truss-web heavy-duty landing mat which was subsequently service tested at Dyess Air Force Base. Approximately 110,000 sq ft of the heavy-duty landing mat were extruded by Taber Metals using die number 10718. The same die was utilized to provide the twelve extrusions for port repair evaluations.

184. The extrusions are one-piece multihollow 6061-T6 aluminum alloy panels, 1-1/2 in. thick, 2 ft wide, and 24 ft long. The panels interlock along the sides by means of a hinge-type male/female connectors, the

components of which are integral parts of the panel extrusion. The average weight of an extrusion (without end connectors) is 5.74 psf. A view of a 24-ft-long truss web extrusion is shown in Figure 63.

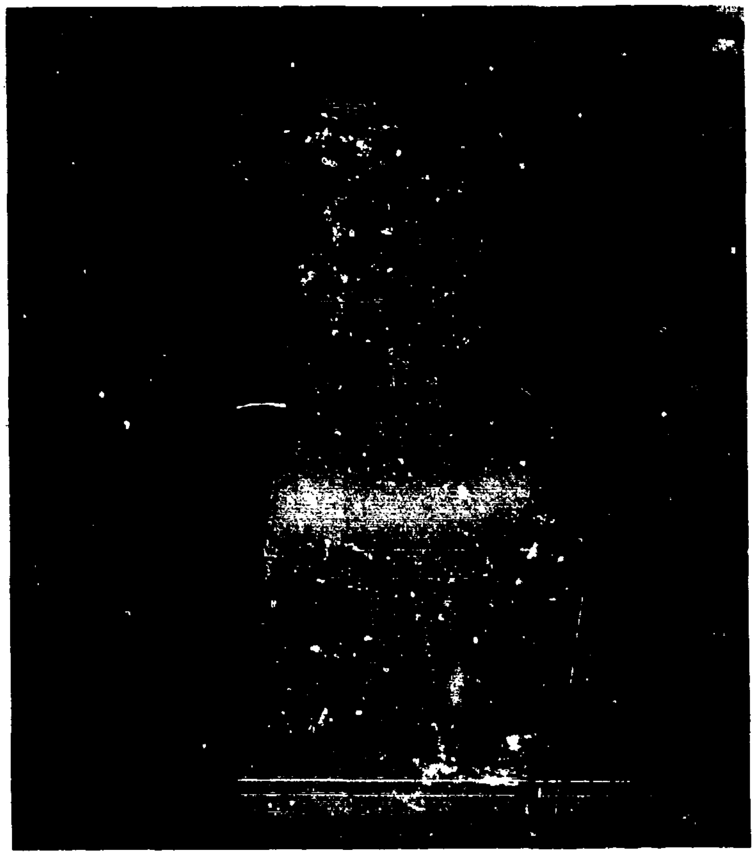


Figure 63. View of truss web extrusion

Test Sections and Equipment

185. Test sections were constructed in an outside environment on a test site previously used for airfield bomb crater repair evaluations. The repaired airfield crater consisted of an open-graded crushed limestone placed in grout. Due to problems with the extrusion anchor bolts pulling out of the grout test area during Tests 1 and 2 traffic, the test section was moved to another airfield repair test area consisting of concrete, and remaining tests were conducted.

186. Test sections were constructed by excavating the desired 4-, 6-, and 8.4-ft diam craters in grout or concrete to a depth of 5 to 6 in. The

grouted or concrete craters were excavated with air-powered jack hammers. These craters simulate the bomb damage on concrete pier/wharf deck structures.

187. Test 1, 2, and 3 test sections consisted of extrusion covered craters of 4-, 6-, and 8.4-ft diam, respectively (Plates 1-3). The required number of 24-ft-long extrusions was cut in half with an electric circular saw equipped with a carbide tip blade. The 12-ft-long extrusions were used in Test 1 and 2. Twenty-four foot long extrusions were used in Test 3. Extrusions were placed over the craters (Figure 64) and anchored into the grout or concrete along the edges with anchor bolts.

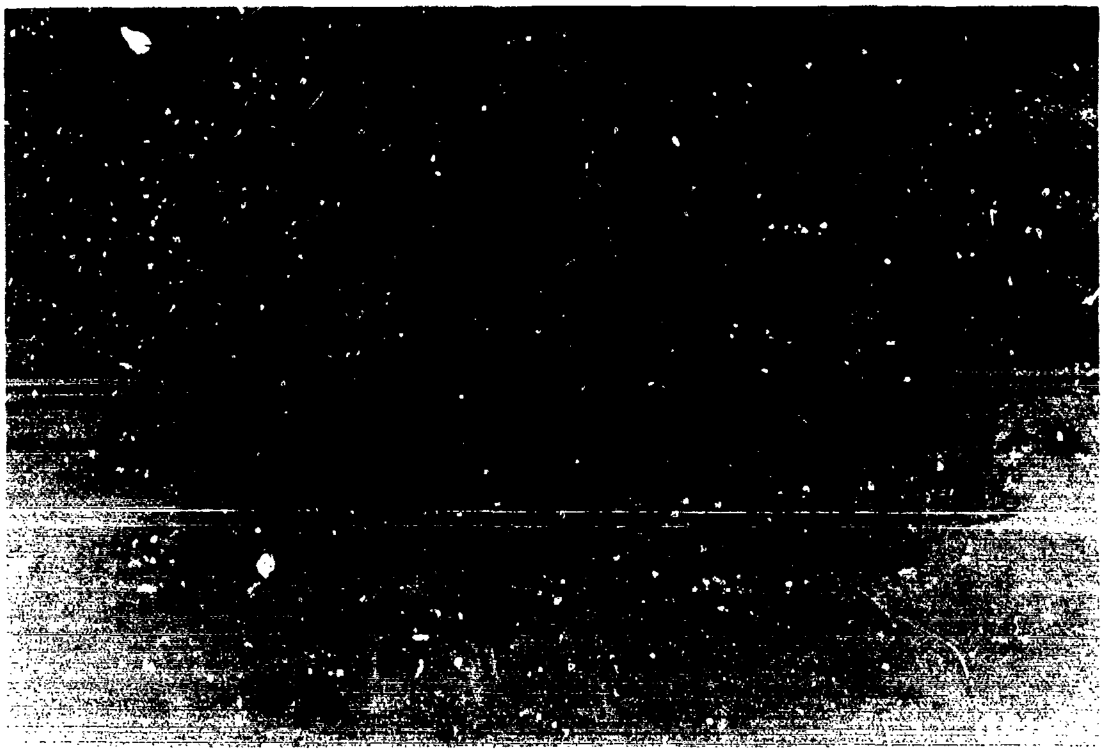


Figure 64. Typical test section of partially covered crater

188. A specially designed single-wheel test cart (Figure 65) loaded to 43,900 lb was used in the traffic tests for Tests 1, 2, and 3. The load wheel had a 20.00-20, 22-ply tire inflated to 125 psi. These load characteristics represent a loaded Belotti straddle carrier (Appendix A) used for container handling on a pier/wharf. Single-lane traffic was applied along the center of each crater. Traffic of 1,000 passes was applied to simulate the anticipated number of passes to unload a containership with an average of 1,000 containers.

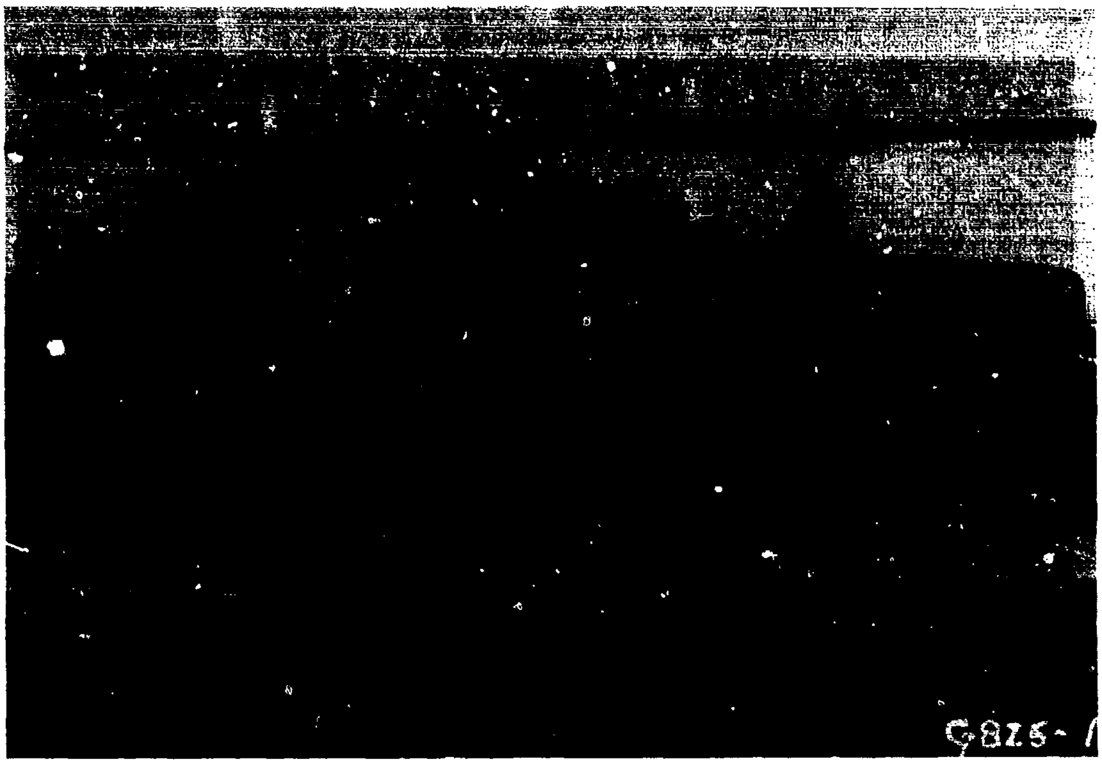


Figure 65. Test vehicle with 43,900-lb single-wheel load on 20.00-20 tire inflated to 125 psi

189. A test section layout for Tests 4, 5, and 6 is shown in Plate 4. The extrusion covered craters of 4-, 6-, and 8.4-ft diam were trafficked with a 16,450-lb dual-wheel load (Figure 66). The loaded dual-wheel had 12.00 by 20, 16-ply tires inflated to 95 psi. These load characteristics represent the maximum load of a M878 yard tractor/M872 trailer (Figure 67) used for container transfer from wharfside to storage yard. The maximum loaded axle on the tractor/trailer (Appendix A) is the rear axle of the M878 yard tractor. Single-lane traffic of 1,000 passes was applied along the center of each crater in Test 4, 5, and 6.

Extrusion Assembly and Placement

190. The 12-ft-long truss-web extrusions were placed across the cratered test section by two individuals under the direction of a foreman. Four individuals and a foreman are necessary during the placement of the heavier 24-ft-long extrusions. Assembly of the extrusions on the test section was

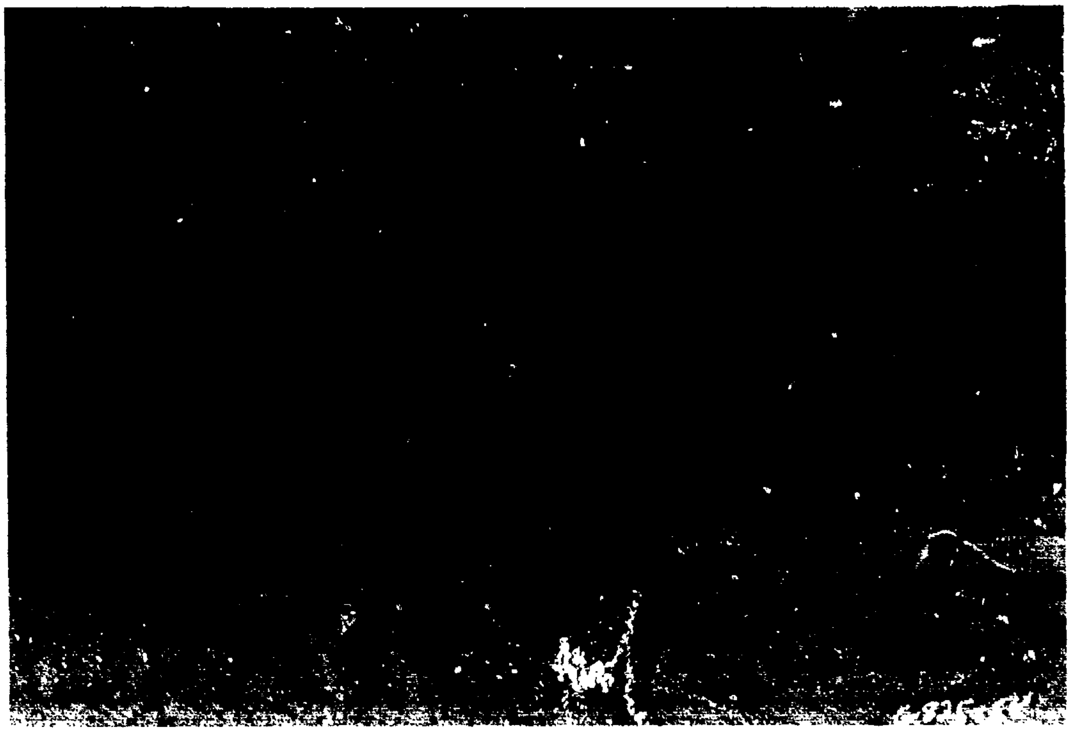


Figure 66. Test vehicle with 16,450-lb dual-wheel load on 12.00 by 20 tires inflated to 95 psi

Figure 67. View of a M878 yard tractor with M872 trailer

accomplished by hinging the female connector onto the male connector and dropping the extrusion into position.

191. Equipment and accessories for anchoring the extrusions to the underlying concrete are shown in Figure 68. The electric power source for operating the two electric drills was a 3,500-w gas-powered portable electric generator. A 1/2-in. electric drill equipped with a 7/8-in. drill bit was used to drill holes in the aluminum extrusions. An electric hammer drill with a 5/8-in. by 12-in.-long carbide percussion drill bit was used to drill holes into the concrete.

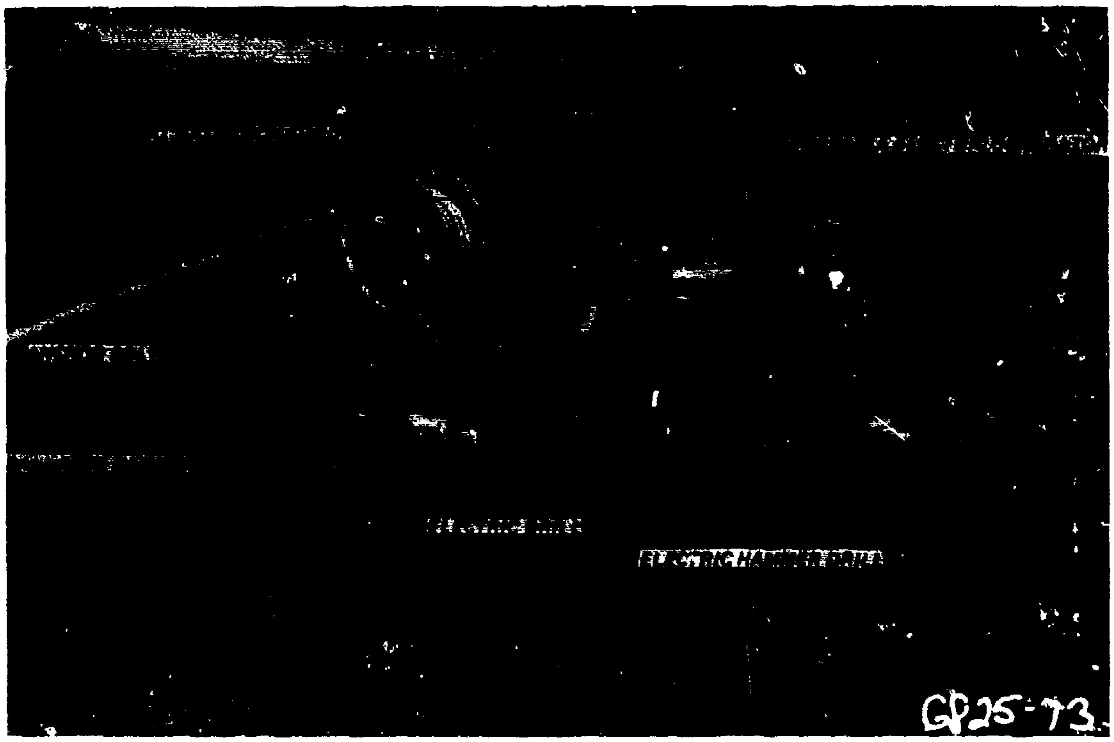


Figure 68. Equipment and accessories for anchoring extrusions to concrete

192. After the extrusions were placed, the extrusions were anchored to the concrete with 5/8-in.-diam by 7-in.-long WEJ-IT concrete anchor bolts (Figure 69). Anchor bolts were placed through 7/8-in. holes in the extrusions and into holes drilled into the concrete. Holes in the extrusions should be predrilled prior to extrusion placement over the damaged hole to be repaired. The following procedure should be used for drilling and installing anchor bolts in concrete.

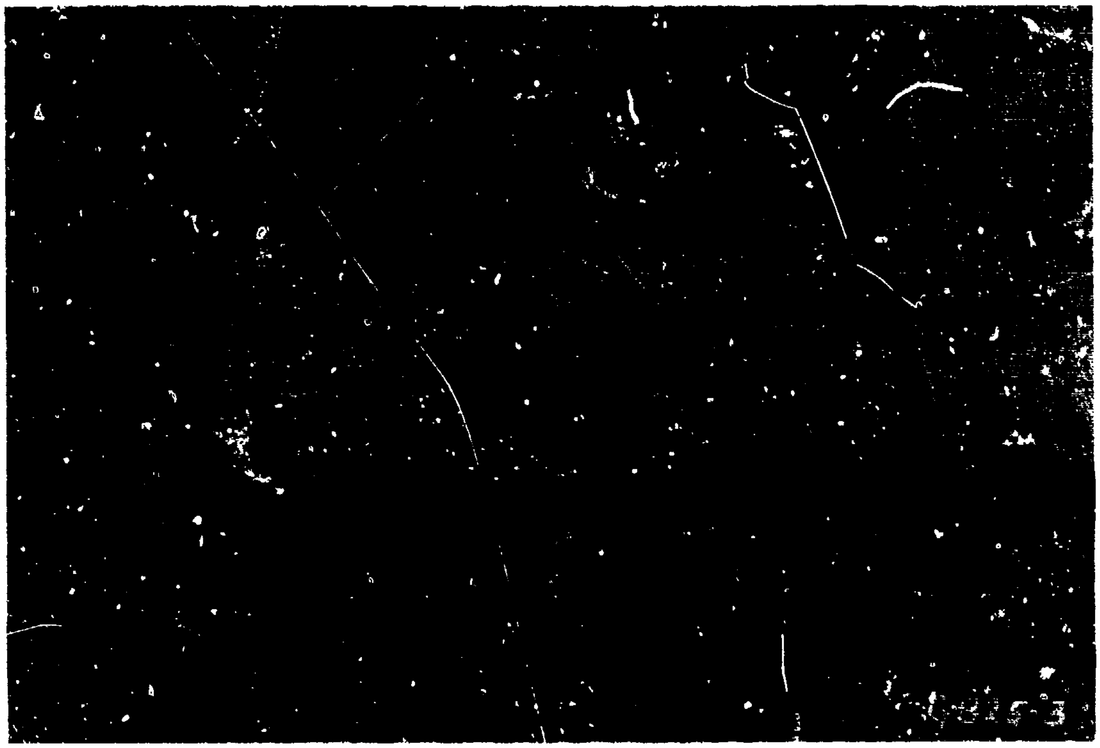


Figure 69. Anchor bolt used to anchor the extrusions to concrete

a. Center the electric hammer drill and bit through the predrilled extrusion hole. Drill the hole perpendicular to the concrete surface. Do not ream the hole or allow the drill to wobble.

b. Drill the hole 7 to 8 in. deep into the concrete.

c. Clean the hole using compressed air or use a long tube and blow through the tube until all dust or loose material is removed.

d. Assemble the washer and nut on the anchor bolt so the nut protrudes slightly beyond the thread.

e. Tap the anchor through the extrusion and into the hole with a ball peen hammer. Make sure the nut rests solidly against the extrusion.

f. Tighten the nut with a ratchet.

The total time required to drill four holes in an extrusion and concrete, place, and anchor each extrusion by a crew of four was an average of 30 min.

Traffic Tests

Test 1

193. An overall view of the test section prior to traffic is shown in Photo 1. Spacing of the anchor bolts placed in the grout/limestone surfacing is shown in Plate 1. No damage occurred to the extrusions after 1,000 single-lane passes of the 43,900-lb single-wheel load (Photo 2). It was noted that five of the eight anchor bolts were pulled upward from the grout/limestone surface after 1,000 passes. Photo 3 shows a 1-1/4-in. deflection measured with a 4-ft straightedge placed parallel to the direction of traffic across the 4-ft crater. Deflection measurements with the load wheel centered over the crater are summarized in Table 21. Permanent set measured with a 4-ft straightedge placed across the crater center and parallel to the dir ction of traffic was negligible after 1,000 passes of traffic. Level readings of cross section (perpendicular to the direction of traffic) and profile (parallel to traffic) were taken prior to and at the conclusion of traffic to measure permanent deformation of the extrusions. The maximum change in both cross section (Plate 5) and profile (Plate 6) measurements from the beginning to 1,000 passes of traffic was 0.3 in. The four 12-ft-long extrusions were removed from the test section and the crater size was increased to 6-ft in diameter for Test 2 traffic tests.

Table 21

Deflection Measurements

Test No.	Crater Diameter, ft	Number of Passes 0	500	1,000
1	4	1-1/8	1-1/8	1-1/4
2	6	1-9/16	1-9/16	1-5/8
3	8.4	3-3/8	3-7/8	3-15/16

Note: Deflection measurements are in inches.

Test 2

194. Four 12-ft-long extrusions were placed over the 6-ft diam crater and were anchored along the edges with anchor bolts as in Test 1. The test section prior to traffic is shown in Photo 4 and after 72 passes in Photo 5.

Six of the eight anchor bolts pulled upward after 72 passes. Since the bolts were not providing proper anchorage, the number of bolts were increased to two anchor bolts per extrusion end as shown in Photo 6. Spacing of the anchor bolts is shown in Plate 2. After 160 passes, 10 of the 16 anchor bolts were either loose or pulled upward (Photo 7). At this pass level it was evident that the grout test area was not a representative test site for providing adequate bond for the anchor bolts. Another test site was located with concrete as a surfacing to anchor and test the extrusions (Photo 8). The extrusions were removed from the grout test site and placed on the concrete site. Each end of the extrusions was anchored with two bolts. After the extrusions were anchored, traffic was continued. The extrusions withstood 1,000 passes of traffic without damage (Photo 9). The anchorage system performed adequately during the 160 to 1,000 passes of traffic. Photo 10 shows a 1-5/8-in. deflection measured with a 6-ft straightedge placed parallel to the direction of traffic across the 6-ft crater. Deflection measurements with the load wheel centered over the crater are summarized in Table 21. Permanent set measured with a 6-ft straightedge placed across the crater center and parallel to the direction of traffic was 3/16 in. after 1,000 passes. The maximum change in both cross section (Plate 5) and profile (Plate 6) measurements from 160 to 1,000 passes of traffic was 0.1 in. The extrusions were removed from the test section and the crater size was increased to 8.4 ft in diameter for Test 3 traffic tests.

Test 3

105. In order to reduce the pullout force on the anchor bolts, the 8.4-ft diameter crater was covered with six extrusions, each 24-ft long. Spacings of the anchor bolts are shown in Plate 3. An overall view of the test section prior to traffic is shown in Photo 11. Visual deflection occurred as the load cart moved back and forth across the test section; however, no surface damage was noticed after 1,000 passes. The anchor bolts performed adequately during the traffic tests. Photo 12 shows a 3-15/16-in. deflection measured with a 8.4-ft straightedge placed parallel to the direction of traffic. Deflection measurements across the 8.4 ft crater with the load wheel centered over the crater are summarized in Table 21. Permanent set of 7/8 in. was measured with a 8.4-ft straightedge placed across the crater center and parallel to the direction of traffic (Photo 13). The maximum

change in both cross section (Plate 5) and profile (Plate 6) measurements from the beginning to 1,000 passes of traffic was 0.8 in.

Test 4, 5, and 6

196. The test section used in Test 3 was left in place, and craters of 4 ft and 6 ft in diameter were excavated with their centers in line with the Test 3 crater so all three covered craters could be trafficked at the same time. The two additional craters were covered with extrusions and anchored. A layout of the test section for Tests 4, 5, and 6 is shown in Plate 4. An overall view of the test section prior to traffic is shown in Photo 14. No damage occurred to the extrusions on either test after 1,000 single-lane passes of the 16,450-lb dual-wheel load (Photo 15), and the anchorage system preformed without any problems. Deflections measured with a straightedge placed parallel to the direction of traffic across the 4-, 6-, and 8.4-ft craters were 1/4, 7/8, and 2-1/2 in., respectively (Photos 16, 17, and 18). Deflection measurements with the load wheel centered over the various sized craters are summarized in Table 22. Permanent set measurements were 0, 1/16, and 7/8 in. across the 4-, 6-, and 8.4-ft craters, respectively, after 1,000 passes of traffic. Cross sections (Plate 7) and profile (Plate 8) measurements show only small deformations from the beginning to 1,000 passes of traffic.

Table 22

Deflection Measurements

Test No.	Crater Diameter, ft	Number of Passes: 0	500	1,000
4	4	1/4	1/4	1/4
5	6	13/16	7/8	7/8
6	8.4	2-1/4(1-5/8)	2-1/2(1-7/8)	2-1/2(1-7/8)

Note: Deflection measurements are in inches. The numbers in parentheses for Test 6 represent the actual deflection minus 5/8-in. permanent set resulting from Test 3.

Extrusion Test Results

197. Traffic capability results of the truss web aluminum extrusions indicate that the material will sustain 1,000 passes each of a 43,900-lb single-wheel load and a 16,450-lb dual-wheel load when placed over craters up to 8.4 ft in diameter. The anchorage system also performed sufficiently during traffic tests when two anchor bolts per extrusion end were placed in concrete.

PART V: STORAGE AREA REPAIR

Introduction

198. The repair of bomb damaged storage areas and other facility pavements at ports is an essential part of a war effort required to keep supplies and equipment moving forward. The purpose of this part is to determine through a literature study those repair techniques that could be applicable to the repair of pavements at ports. A significant amount of work has been accomplished in the area of repair of bomb damaged airfield pavements. This work has involved numerous testing programs directed at repair of runways used by the F-4 and C-141 aircraft. The bibliography presented in Appendix E lists numerous reports that were surveyed to provide data for the basis of selecting materials and techniques for pavement repair. The WES, Corps of Engineers, US Navy, and US Air Force have been responsible for the major efforts in repair of bomb damaged runways. The WES conducted a series of tests on various repair techniques and materials, and the results of these investigations are shown in Tables 23 to 27. In addition, the results of selected tests conducted by or for the Air Force are shown in Table 28. The test results shown are those which have been subjected to aircraft type loadings. Generally, the C-141 load was 144,000 lb on four wheels representing one landing gear of the C-141 cargo aircraft, and the F-4 load ranged from 25,000 to 30,500 lb on a single wheel representing one landing gear of the F-4 fighter aircraft. These loadings are in the same range as those anticipated at ports; therefore, the traffic test results conducted for runways are considered applicable to pavements at ports. The requirements for ports are in some ways different from the requirements for airfield pavements. Smoothness and time requirements for ports are not critical as requirements for airfield pavements. This makes some repair materials and techniques satisfactory for port pavements that are not considered acceptable for airfields.

199. The port vulnerability study presented in Part II of this report estimates the physical damage and the expected number of bomb hits on a civilian container port exposed to an aerial general purpose bomb threat. Based on the study, numerous craters of 44 ft in true diameter (Figure 70) produced by 500-lb bombs are anticipated in storage areas adjacent to piers/wharves.

Table 23

Test Series 1

Report Referenced	Crater Number	Material Tested	Thickness in.	Aircraft	Aircraft Characteristics: Wheel Configuration	Wheel Spacing in.	Test Load lb	Tire Pressure psi	Contact Area sq in.	Tire Size	Tire Ply Rating	Number of Passes Applied: Lane 1	Lane 2	Soil Strength	Performance of Material Tested	Hours After Construction Until the Surface Is Usable
Cooksey 1981	1	Crushed limestone compacted well-graded	18	C-141 F-4C	Twin-tandem single	32.5 × 48	144,000 25,000	185 250	208 111	44 × 16 30.0 × 11.5	28 24	5,100 760	5,100 0	K = 167 on washed gravel	Satisfactory with some remedial repair	0
	2	Open-graded aggregate placed in hole and pressure grouted	16	C-141 F-4C	Twin-tandem single	32.5 × 48	144,000 25,000	185 250	208 111	44 × 16 30.0 × 11.5	28 24	5,100 760	5,100 0		Satisfactory	8
	3	Open-graded aggregate placed in grout	18	C-141 F-4C	Twin-tandem single	32.5 × 48	144,000 25,000	185 250	208 111	44 × 16 30.0 × 11.5	28 24	5,100 760	5,100 0	K = 156 on washed gravel	Satisfactory	8
	4	Conventional portland cement concrete	15	C-141 F-4C	Twin-tandem single	32.5 × 48	144,000 25,000	185 250	208 111	44 × 16 30.0 × 11.5	28 24	5,100 760	5,100 0	K = 182 on washed gravel	Satisfactory	24
	5	Conventional portland cement concrete	15	C-141 F-4C	Twin-tandem single	32.5 × 48	144,000 25,000	185 250	208 111	44 × 16 30.0 × 11.5	28 24	5,100 760	5,100 0	K = 182 on rubble	Satisfactory	24
	6	Asphaltic concrete over crushed stone	3 9	C-141 F-4C	Twin-tandem single	32.5 × 48	144,000 25,000	185 250	208 111	44 × 16 30.0 × 11.5	28 24	5,100 760	5,100 0	CBR = 125	Satisfactory with some densification	24

Table 24

Test Series 2

Report Referenced	Crater Number	Material Tested	Aircraft	Aircraft Characteristics: Wheel Configuration	Wheel Spacing in.	Test Load lb	Tire Pressure psi	Contact Area sq in.	Tire Size	Tire Ply Rating	Passes	Performance of Material Tested	Hours After Construction Until the Surface is Usable*
Alford and Hammitt 1985a	1 (Lane 1)	Single surface treatment over 18 in. of crushed limestone	C-141	Twin-tandem	32.5 × 48	144,000	185	208	44 × 16	28	1,360	Satisfactory	4
	1 (Lane 2)	Double surface treatment over 18 in. of crushed limestone	C-141	Twin-tandem	32.5 × 48	144,000	185	208	44 × 16	[illegible]	850	Satisfactory - however asphalt was being tracked from the repair onto the regular pavement	4
	2 (Lane 1)	12 in. of AC over 4 ft of gabions	C-141	Twin-tandem	32.5 × 48	144,000	185	208	44 × 16	28	1,360	Unsatisfactory for C-141 due to rutting of 2 in. after 850 passes	24
	2 (Lane 2)	12 in. of AC over 4 ft of gabions	C-141	Twin-tandem	32.5 × 48	144,000	185	208	44 × 16	28	850	Unsatisfactory for C-141 due to rutting of 3 in. after 850 passes	24
	3 (Lane 1)	12 in. of AC over 6 in. of sand and mat	C-141	Twin-tandem	32.5 × 48	144,000	185	208	44 × 16	28	510	Unsatisfactory due to consolidation	24
	3 (Lane 2)	12 in. of AC over 6 in. of sand and mat	C-141	Twin-tandem	32.5 × 48	144,000	185	208	44 × 16	28	510	Unsatisfactory due to consolidation	24

* The time listed in this column applies only if the temperature is 80°F or below.

Table 25

Test Series 3

Report Referenced	Crater Number	Material Tested	Aircraft	Aircraft Characteristics: Wheel Configuration	Wheel Spacing in.	Test Load lb	Tire Pressure psi	Contact Area sq in.	Tire Size	Tire Ply Rating	Passes	Soil Strength	Performance of Material Tested	Hours After Construction Until the Surface is Usable
Alford and Hammitt 1985a	1 (Lane 1)	T-17 membrane over 18 in. of WES blended material over 8.5 ft of washed gravel	F-4C	Single		27,000	243	111	30.0 × 11.5	24	1	CBR = 2.7	Unsatisfactory - 5 in. rut after first pass	0
	1 (Lane 2)	T-17 membrane over 18 in. of WES blended material over 8.5 ft of washed gravel	C-141	Twin-tandem	32.5 × 48	144,000	185	208	44 × 16	28	1	CBR = 2.7	Unsatisfactory - 5 in. rut after one pass	0
	2 (Lane 1)	18 in. of WES blended material stabilized with 8 percent (by weight) of portland comentover 8.5 ft of washed gravel	F-4C C-141	Single Twin-tandem	32.5 × 48	27,000 144,000	243 185	111 208	30.0 × 11.5 44 × 16	24 28	550 2,380	K = 232 on base	Satisfactory - average 1.0 in. deformation	36
	2 (Lane 2)	18 in. of WES blended material stabilized with 8 percent (by weight) of portland cement over 8.5 ft of washed gravel	C-141	Twin-tandem	32.5 × 48	144,000	185	208	44 × 16	28	2,380	K = 232 on base	Satisfactory - average 1.5 in. deformation	36
	3 (Lane 1)	18 in. of large crushed stone grouted	F-4C C-141	Single Twin-tandem	32.5 × 48	27,000 144,000	243 185	111 208	30.0 × 11.5 44 × 16	24 28	550 680	K = 158 on base	Satisfactory	12
	3 (Lane 2)	18 in. of large crushed stone grouted	C-141	Twin-tandem	32.5 × 48	144,000	185	208	44 × 16	28	680	K = 158 on base	Satisfactory	12
	4 (Lane 1)	12 in. of PCC over 9.0 ft of washed gravel	F-4C C-141	Single Twin-tan em	32.5 × 48	27,000 144,000	243 185	111 208	30.0 × 11.5 44 × 16	24 28	550 2,380	K = 200 on base	Satisfactory with minor transverse cracks	24
	4 (Lane 2)	12 in. of PCC over 9.0 ft of washed gravel	C-141	Twin-tandem	32.5 × 48	144,000	185	208	44 × 16	28	2,380	K = 200 on base	Satisfactory with transverse cracks and one longitudinal crack	24
	5 (Lane 1)	14 in. of crossties (2 layers) over washed gravel base	F-4C C-141	Single Twin-tandem	32.5 × 48	27,000 144,000	243 185	111 208	30.0 × 11.5 44 × 16		550 1,530	K = 188 on base	Unsatisfactory due to roughness	0
	5 (Lane 2)	14 in. of crossties (2 layers) over washed gravel base	C-141	Twin-tandem	32.5 × 48	144,000	185	208	44 × 16		1,530	K = 188 on base	Unsatisfactory due to roughness	0

Table 26

Test Series 4

Report Referenced	Crater Number	Material Tested	Aircraft	Aircraft Characteristics: Wheel Configuration	Wheel Spacing in.	Test Load lb	Tire Pressure psi	Contact Area sq in.	Tire Size	Tire Ply Rating	Passes	Soil Strength	Performance of Material Tested	Hours After Construction Until the Surface is Usable
Alford and Hammitt 1985b	1 (Lane 1)	14 in. of an open-graded crushed limestone placed in a neat grout* over 8 ft 10 in. of washed gravel	F-4C C-141	Single Twin-tandem	32.5 × 48	27,000 144,000	243 185	111 208	30.0 × 11.5 44 × 16	24 28	550 3,400	K of washed gravel = 370	Satisfactory	10
	1 (Lane 2)	14 in. of an open-graded crushed limestone placed in a neat grout* over 8 ft 10 in. of washed gravel	C-141	Twin-tandem	32.5 × 48	144,000	185	208	44 × 16	28	3,400	K of washed gravel = 370	Satisfactory	10
	2 (Lane 1)	18 in. of an open-graded crushed limestone placed in a neat grout* over 8 ft 6 in. of washed gravel	F-4C C-141	Single Twin-tandem	32.5 × 48	27,000 144,000	243 185	111 208	30.0 × 11.5 44 × 16	24 28	550 3,400	K of washed gravel = 455	Satisfactory	12
	2 (Lane 2)	18 in. of an open-graded crushed limestone placed in a neat grout* over 8 ft 6 in. of washed gravel	C-141	Twin-tandem	32.5 × [illegible]	144,000	185	208	44 × 16	28	3,400	K of washed gravel = 455	Satisfactory	12
	3 (Lane 1)	Bituminous seal coat over 15 in. of sandy gravel stabilized with 8 percent by weight of Type III, high early-strength portland cement	F-4C C-141	Single Twin-tandem	32.5 × 48	27,000 144,000	243 185	111 208	30.0 × 11.5 44 × 16	24 28	550 3,400	K of washed gravel = 357	Unsatisfactory due to rutting and breakup of the stabilized sandy gravel	36
	3 (Lane 2)	Bituminous seal coat over 15 in. of sandy gravel stabilized with 8 percent by weight of Type III, high early-strength portland cement	C-141	Twin-tandem	32.5 × 48	144,000	185	208	44 × 16	28	3,400	K of washed gravel = 357	Unsatisfactory due to rutting and breakup of the stabilized sandy gravel	36
	4 (Lane 1)	2 in. neat-grout cap placed over 14 in. of well-graded crushed limestone	F-4C C-141	Single Twin-tandem	32.5 × 48	27,000 144,000	243 185	111 208	30.0 × 11.5 44 × 16	24 28	550 3,400	K of washed gravel = 476 CBR of crushed stone = 97	Unsatisfactory due to spalling of the grout cap	8
	4 (Lane 2)	2 in. neat-grout cap placed over 14 in. of well-graded crushed limestone	C-141	Twin-tandem	32.5 × 48	144,000	185	208	44 × 16	28	3,400	K of washed gravel = 476 CBR of crushed stone = 97	Unsatisfactory due to spalling of the grout cap	8
	5 (Lane 1)	12 in. of well-graded crushed limestone overlayed with a double surfaced treatment	F-4C C-141	Single Twin-tandem	32.5 × 48	27,000 144,000	243 185	111 208	30.0 × 11.5 44 × 16	24 28	550 2,074	K of washed gravel = 278 CBR of crushed stone = 100 crushed	Unsatisfactory due to rapid cooling of asphalt which prevented seating of aggregate	4
	5 (Lane 2)	12 in. of well-graded crushed limestone overlayed with a double surfaced treatment	C-141	Twin-tandem	32.5 × 48	144,000	185	208	44 × 16	28	3,400	K of washed gravel = 278 CBR of crushed stone = 100	Unsatisfactory due to rapid cooling of asphalt which prevented seating of aggregate	4

* Compressive strength of neat grout was 590 psi, and flexural strength was 80 psi.

Table 27
Test Series 5

Report Referenced	Crater Number	Material Tested	Aircraft	Aircraft Characteristics: Wheel Configuration	Wheel Spacing in.	Test Load lb	Tire Pressure psi	Contact Area sq in.	Tire Size	Tire Ply Rating	Number of Coverages Applied: Lane 1	Lane 2	Soil Strength	Performance of Material Tested*	Hours After Construction Until the Surface is Usable
Shoenberger and Hammitt, (in preparation)	1	24 in. thick grout with 3 in. rock added over T-17 waterproof membrane over 96 in. selected debris	F-4C C-141	Single Twin-tandem	32.5 × 48	27,000 144,000	265 185	111 208	30.0 × 11.5 44 × 16	24 28	300 950	150 1,000	Not obtained	Satisfactory	16
	2	1/2 in. FRP (fiber reinforced polyester) over yers 3 layers of 8 in. sand grids (plastic) over 96 in. of selected debris	F-4C C-141	Single Twin-tandem	32.5 × 48	27,000 144,000	265 185	111 208	30.0 × 11.5 44 × 16	24 28	300 820	150 1,000	K = 294 on selected debris	Satisfactory	0
	3	Lane 1 consisted of 2 items. Item 1 was 12 in. of crushed stone over 18 in. of low density foam over 90 in. of selected debris.* Item 2 was 12 in. of high density foam over 18 in. of low density foam	F-4C C-141	Single Twin-tandem	32.5 × 48	27,000 144,000	265 185	111 208	30.0 × 11.5 44 × 16	24 28	300 820		K = 192 on selected debris	Satisfactory, only low density block exhibited distress	0
	3	Lane 2 consisted of 24 in. of crushed limestone with 4 layers of Netlon reinforcing placed below each 6 in. lift over 96 in. of selected debris	F-4C C-141	Single Twin-tandem	32.5 × 48	27,000 144,000	265 185	111 208	30.0 × 11.5 44 × 16	24 28		150 1,000	K = 159 on selected debris	Satisfactory with moderate rutting	0

* After 150 passes of the F-4C load, the repair was reconstructed with the low density foam being removed and used as the surfacing layer over crushed limestone.

Table 28

Traffic Tests Conducted for US Air Force and US Navy

Report Referenced	Lane	Material Tested	Aircraft	Wheel Configuration	Wheel Spacing in.	Test Load lb	Tire Pressure psi	Contact Area sq in.	Tire Size	Tire Ply Rating	Passes	Strength Data	Performance of Material Tested
				Aircraft Characteristics									
Springston 1983	—	3/8 in. fiberglass-reinforced polyester (FRP) membrane over 24 in. of crushed stone over approximately 6 ft of selected debris	F-4	Single	—	27,000	265	102	30.0 × 7.7	18	1,440*	Selected debris CBR = 3 Base course CBR > 30	Satisfactory
Springston 1983	—	3/8 in. fiberglass-reinforced polyester (FRP) membrane over 12 in. of crushed stone over 1/8 in. fiberglass-reinforced polyester (FRP) membrane over approximately 3 ft of selected debris	F-4	Single	—	27,000	265	102	30.0 × 7.7	18	1,440*	Selected debris CBR = 3 Base course CBR > 30	Satisfactory
Wang 1975	—	Liquid epoxy resin was placed on 12 in. of 3/4 in. uniform gravel over 4 in. of concrete sand and 50 in. of silt	F-4	Single	—	30,500	280	109	30.0 × 7.7	18	100	CBR of silt = 6	Satisfactory
Rollings 1975	—	Class 60 trackway	F-4	Single	—	27,600	267	103	30.0 × 7.7	18	100	CBR of debris = 9	Satisfactory
Rollings 1980	—	AM-2 mat over a 6 in. base course of well graded aggregate	F-4	Single	—	27,000	265	102	30.0 × 11.5	24	1,152	CBR of select fill = 59 CBR of clay subgrade = 7	Satisfactory
Hokenson and Rollings 1975	—	AM-2 mat over 19 in. of select fill over clay subgrade	F-4	Single	—	30,000	280	102	30.0 × 11.5	24	51	CBR of select fill = 10.6 CBR of clay subgrade = 7.3	Satisfactory
		AM-2 mat over 12 to 21 in. of select fill over clay subgrade	F-4	Single	—	30,000	280	102	30.0 × 11.5	24	51	CBR of select fill = 47.2 CBR of clay subgrade = 9.0	Satisfactory
Carroll and Sutton 1965	—	AM-2 mat over 4 in. of sand over 12 in. course limestone aggregate over sand clay subgrade	F-4	Single	—	29,000	256	112	30.0 × 11.5	24	16	Clay subgrade CBR = 0.7	Satisfactory
Springston and Claxton 1983	1	1/2 in. FRP over two 8 in. sand filled aluminum grid over 6 in. of course, rounded poorly graded gravel (simulating crater debris)	F-4 C-141	Single Twin-tandem	— 32.5 × 48	27,000 144,000	267 185	100 208	30.0 × 11.5 44 × 16	24 28	547 340	K = 350 pci on gravel	Satisfactory Satisfactory
	2	1/2 in. FRP over two 8 in. filled aluminum grid over 6 ft of course, rounded poorly graded gravel (simulating crater debris)	C-130	Single-tandem	60	38,000	100	400	20 × 20	20	120	K = 350 pci on gravel	Satisfactory

* 49 passes of F-4 aircraft was also applied.

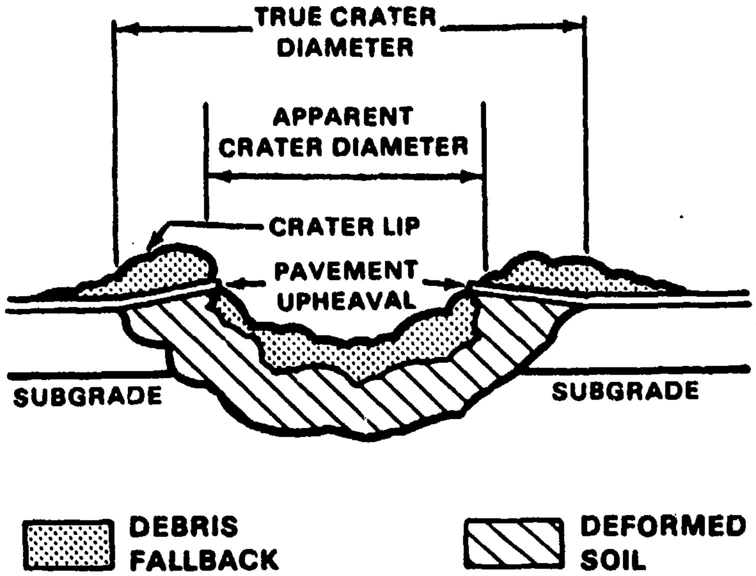


Figure 70. Pavement damage from bomb detonation

Crater repair is necessary to maintain continuous port operations for supplying military troops in the theater of operations.

Traffic Test Summaries

Test series 1

200. Table 23 presents the results of test series 1 conducted by WES (Cooksey 1981). A pavement test section (Figure 71) was constructed consisting of six craters that were filled with various materials to investigate repair techniques. Craters 1, 2, 3, 4, and 6 were partially filled with washed gravel, and Crater 5 was filled with lean clay and rubble. The craters were then capped with crushed limestone, grout, conventional portland cement concrete (PCC), and a flexible pavement. Two traffic lanes were placed on each repair. All test items in Lanes 1 and 2 sustained 5,100 passes of the C-141 gear load and the items in Lane 1 also sustained 760 passes of an F-4C gear load. All repairs performed satisfactorily with the crushed stone considered to be the most promising repair technique because of the speed of construction, low cost, and less sophisticated equipment, and the repaired crater can be used immediately.

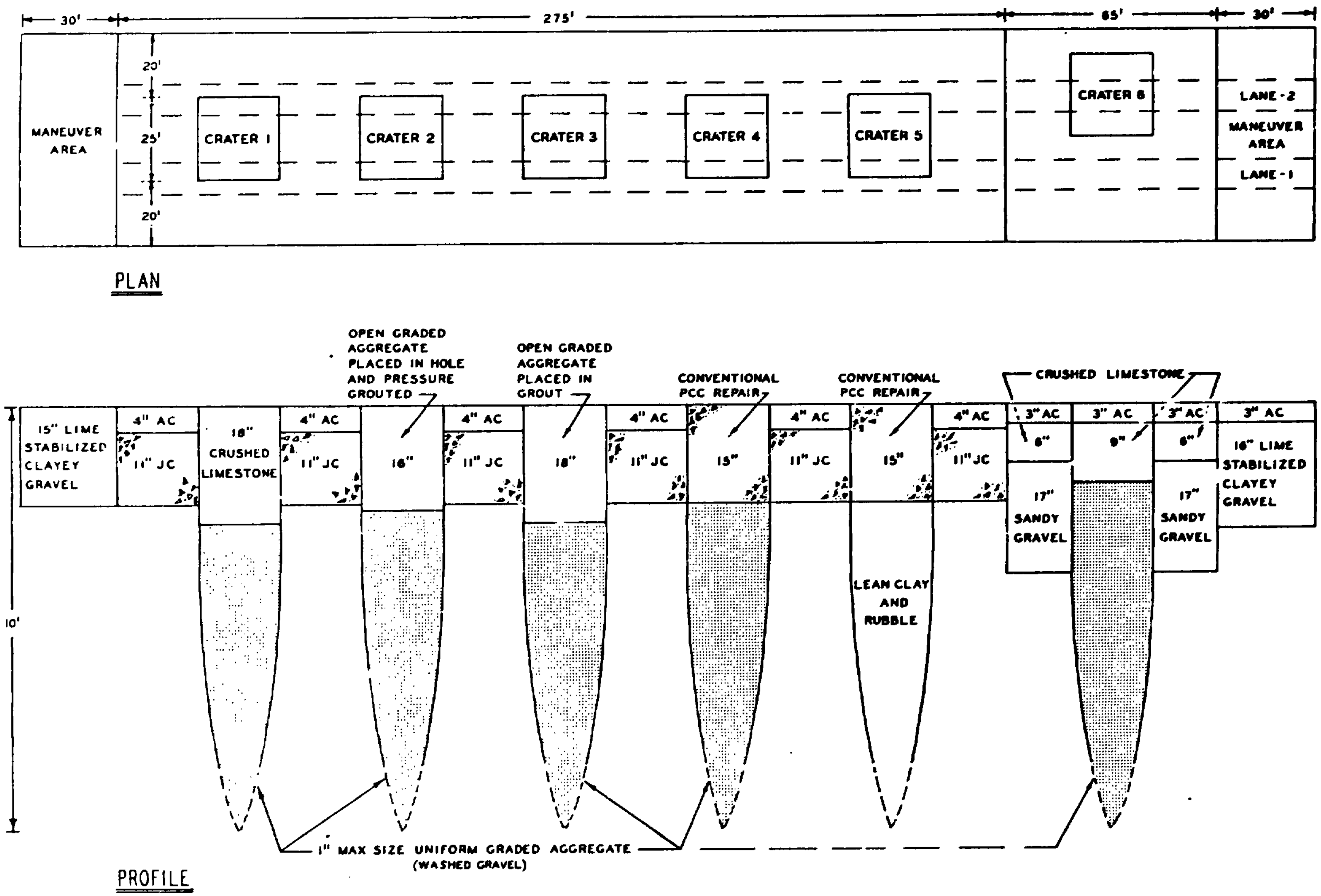


Figure 71. Test section for test series 1

Test series 2

201. Table 24 presents the results of test series 2 (Alford and Hammitt 1985a) which was constructed using Craters 1, 2, and 3 of test series 1 (Figure 72). In Crater 1 a single surface treatment was placed in Lane 1 and a double surface treatment in Lane 2 over the crushed stone from test Section 1. In Crater 2, 12 in. of asphaltic concrete (AC) was placed over 4 ft of gabions. In Crater 3, 12 in. of AC was placed over 6 in. of sand on top of landing mat panels. The surface treatments performed satisfactorily, but would require an asphalt material with fast setting properties. The wire gabions in Crater 2 offer a solution in a wet crater but were not an adequate semipermanent repair for C-141 traffic and therefore would probably not be satisfactory for the heavy loads at ports. In Crater 3, there was significant consolidation due to a lack of density in the sand layer and was therefore not satisfactory.

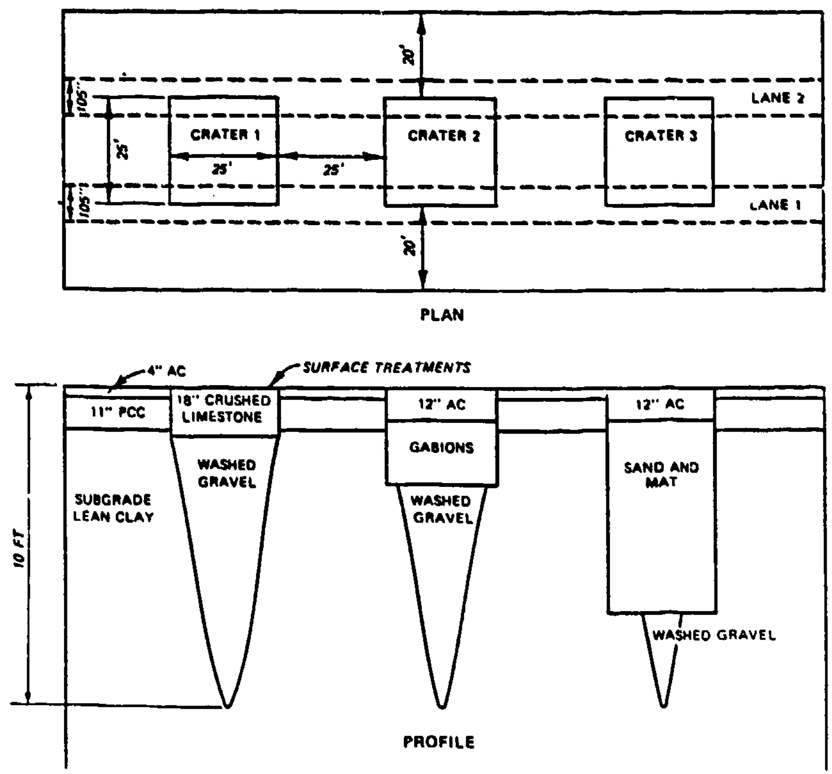


Figure 72. Test section for test series 2

Test series 3

202. Test series 3 (Alford and Hammitt 1985a) was constructed using Craters 1 through 5 from the initial test series (Figure 73). Results of test series 3 are presented in Table 25. This series of tests were conducted using

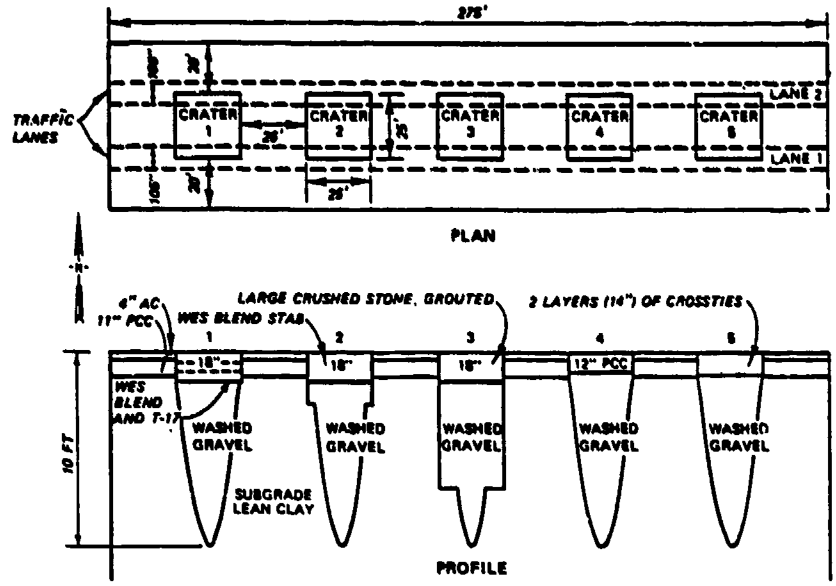


Figure 73. Test section for test series 3

surfacings of T-17 membrane, cement stabilized soil, grouted stone, PCC, and wooden crossties. The T-17 membrane was unsatisfactory for the C-141 and F-4C traffic since 5 in. ruts were produced by the first pass of the landing gear loads. The stabilized soil in Crater 2 performed in a reasonably satisfactory manner, although deformation of 1 to 1.5 in. was produced under traffic. Although this deformation may be unsatisfactory for aircraft, it may be tolerable for ground vehicles in port areas. The repair technique for Crater 3 consisted of using grouted crushed stone. When placed properly, the grouted material will perform in a satisfactory manner. Construction is accomplished by placing grout in the crater and then adding the crushed stone to the grout. The PCC repairs in Crater 4 performed in a satisfactory manner although there was some cracking in Lane 2 under the C-141 traffic. Crossties were used as a surfacing in Crater 5. The roughness that developed in the crossties make them unsatisfactory for use in a runway but may be satisfactory for expedient repair in a road or paved storage area where roughness may not be critical.

Test series 4

203. Table 26 presents the results of test series 4 (Alford and Hammitt 1985b). Test series 4 (Figure 74) consisted of crater repairs using two thicknesses of an open-graded limestone placed in a neat grout, a bituminous seal coat placed over a cement stabilized sandy gravel, a neat grout placed over a crushed limestone, and a double bituminous surface treatment placed

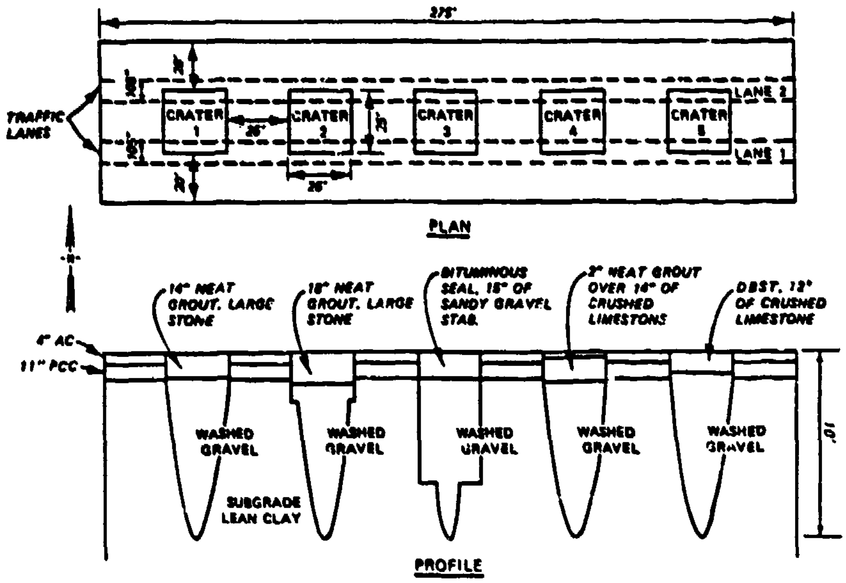


Figure 74. Test section for test series 4

over a crushed limestone. The 14 and 18 in. thicknesses of open-graded crushed limestone placed in the neat grout performed in a satisfactory manner with the 18 in. thickness providing the best performance. However, 18 hr cure time is necessary for best performance of the repair. The bituminous seal coat placed on the cement stabilized sandy gravel in Crater 3 was not satisfactory for crater repair due to rutting and breakup of the stabilized material. The neat grout placed over limestone in Crater 4 spalled and was unsatisfactory as a technique for preventing foreign object damage to aircraft when using crushed stone as a repair. The double bituminous surface treatment was unsatisfactory. The AC-20 cooled too quickly for proper seating of the aggregate into the binder. Also, the amount of asphalt used was somewhat higher than necessary.

Test series 5

204. Table 27 presents test results for test series 5 (Shoenberger and Hammitt, in preparation). These tests were conducted on repairs (Figure 75) consisting of a grout and membrane in Crater 1, fiber reinforced polyester over sand grids in Crater 2, and combinations of foam and crushed stone in Lane 1 of Crater 3 and crushed stone and Netlon (a high density plastic mesh for soil stabilization) in Lane 2 of Crater 3. The thick grout in Crater 1 contained 3-in. rock and was placed over T-17 waterproof membrane. It was trafficked with F-4C and C-141 loadings and performed in a satisfactory

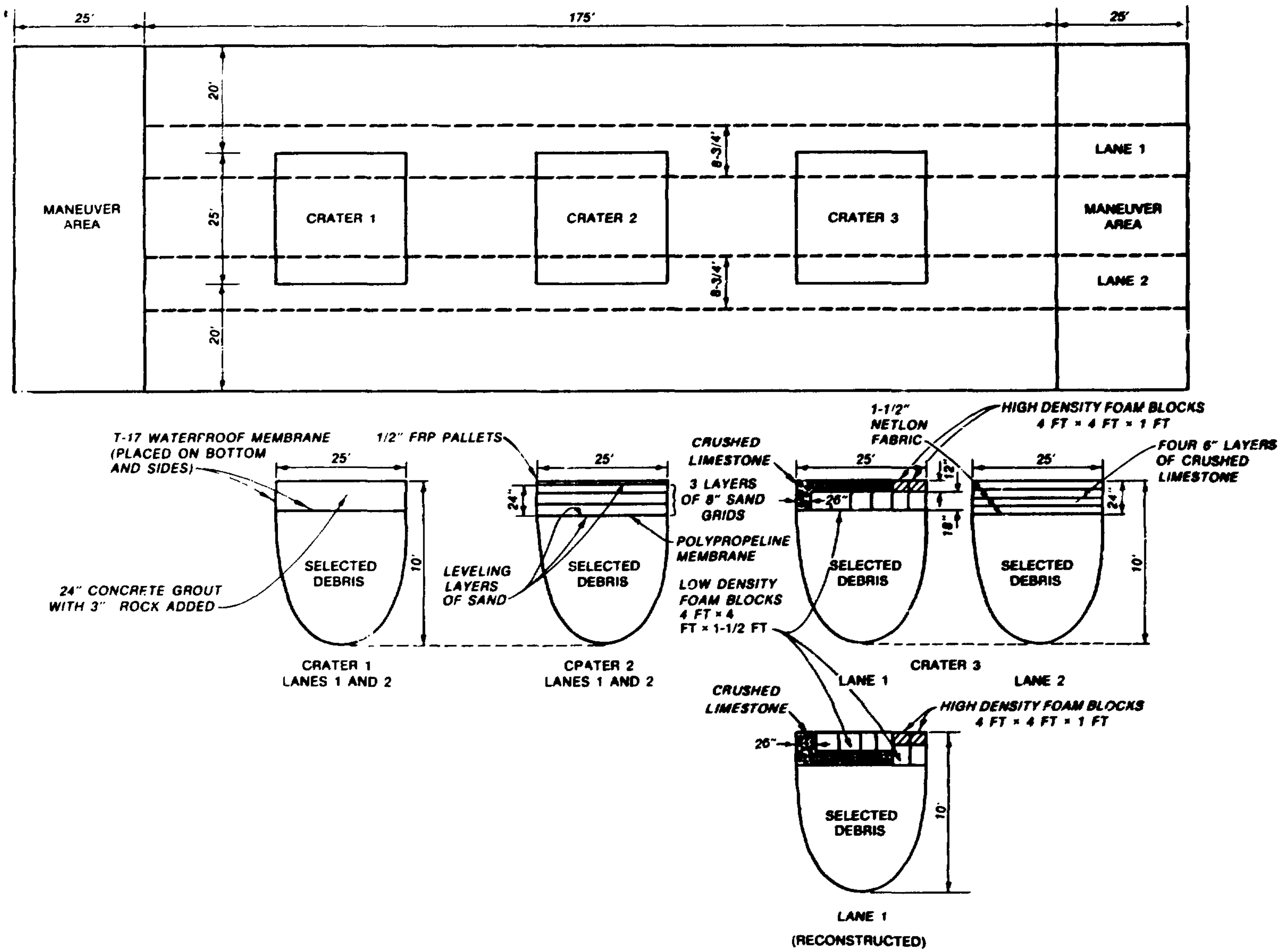


Figure 75. Test section for test series 5

manner. In Crater 2, a 1/2-in. layer of fiber reinforced polyester was placed over three layers of 8 in. plastic sand grids. This repair also performed in a satisfactory manner after being trafficked with the F-4C and C-141 loadings. The repair in Lane 1 of Crater 3 consisted of a high density foam placed over 12 in. of crushed limestone placed over 18 in. of low density foam. This repair technique also supported traffic of the F-4C and C-141 loadings and performed satisfactorily. Lane 2 of Crater 3 consisted of 24 in. of Netlon reinforcing placed below each 6-in. layer of stone. This repair method sustained moderate rutting but was considered a satisfactory repair. Each of these repair techniques performed in a satisfactory manner and should be capable of adaptation to repair of pavements in port areas.

Other traffic tests

205. Results of traffic tests on several repair techniques are shown in Table 28. These data are from several sources and were trafficked with F-4, C-141, and C-130 type traffic. Two of these tests involved the use of fiberglass-reinforced polyester (FRP) as a surfacing. The FRP was placed over 24-in. of crushed stone in one of these tests and over 12 in. of crushed stone which was placed on FRP as a sandwich type construction. Each of these repair methods sustained 1,440 passes of a test load cart with F-4 loading. In addition 49 passes of an F-4 aircraft loaded to 27 kips, and a tire pressure of 265 psi were applied to the repairs. Each of these repairs performed in a satisfactory manner with little or no deformation.

206. Table 28 also presents the results of data where liquid epoxy resin (Furane Plastic Epocast 530 with 946 hardner-cure time 10 to 15 min) was placed on 12 in. of 3/4 in. uniform gravel over 4 in. of concrete sand over 50 in. of silt. An F-4 tire with a test load of 30,500 lb and a tire pressure of 280 psi was used to traffic the repair area. The cure time for the neat resin was 10 to 15 min; however, the cure time for the aggregate/resin system can be expected to be somewhat longer because the aggregates serve as a heat sink and thus retard curing. Two test beams were fabricated during pouring operations at 9 a.m. and were ready for testing at approximately 3 p.m. the same day. The flexure strength of these beams was 1,400 psi and 1,375 psi. This repair successfully withstood 100 passes of the traffic loading without any visible or measurable signs of distress.

207. The results of tests on British Class 60 Trackway are also shown in Table 28. This is an aluminum planking which can be placed over a crater

after backfilling in order to support anticipated loads. The trackway was subjected to 100 passes of an F-4 type loading, and performed in a satisfactory manner. It was recommended for use in airfield bomb damage repair and should also be satisfactory as a repair technique for pavements in port areas.

208. The use of AM-2 landing mats as a surfacing for crater repairs has been the accepted technique for repair of runways for many years. Several tests on AM-2 are presented in Table 28. Various bases were used under the mat including well graded aggregate, select fill, and limestone. In each test the mat performed in a satisfactory manner under traffic of the F-4 type loading. From the standpoint of strength, the AM-2 is capable of supporting anticipated traffic at ports.

209. Additional tests using 1/2 in. FRP are also shown on Table 28. In these tests FRP was placed over aluminum sand grids over 6 ft of rounded aggregate backfill. These tests also performed well as did the FRP in other tests, after being trafficked with the F-4, C-141, and C-130 type loading.

Recommended Repair Methods

210. There are several repair methods for bomb damaged pavements at port facilities that are considered satisfactory. The best repair method is to replace in kind the pavement that was damaged. That is, replace a damaged flexible or rigid pavement with the same materials and the same thicknesses as the original pavement. Where time is not a constraint and materials are available would be the preferred repair method. However, where the situation dictates the use of other repair methods, the following expedient techniques are considered satisfactory for well graded crushed stone, grouted aggregate, PCC, AC, and cement stabilized gravel, respectively.

- a. The use of well graded crushed stone as a repair technique is recommended since it is a fast repair method that provides sufficient strength and is easily maintained. Use of crushed stone requires backfilling the crater, compacting the backfill to at least 85 percent CE 55 compactive effort, and placing 18 in. of well graded crushed stone compacted to at least 95 percent CE 55. Use of a surfacing is not necessary for the purpose of carrying the load but may be needed for waterproofing to resist shear due to turning wheel loads or other reasons where required. The surfacing may be 1/2 in. FRP, bituminous surface treatment, landing mat, or Class 30 trackway.

b. The use of grouted aggregate is also an acceptable method of crater repair. The aggregate may be placed in the crater and pressure grouted or the grout may be placed in the crater and aggregate added to the grout. This type of repair requires about 18 in. of grouted material over the crater backfill compacted to at least 85 percent CE 55 compactive effort.

c. Conventional PCC served as an adequate crater repair technique. This repair requires 15 in. of PCC to be placed over the 85 percent CE 55 compacted backfill. Where a good quality gravel is used to backfill the crater, 12 in. of concrete may be used.

d. The use of an AC surfacing over a crushed stone base course provides satisfactory performance with some densification. It is suggested that this type repair consist of 3 in. of AC and 12 in. of crushed stone to reduce the densification. The crushed stone should be compacted to a minimum of 95 percent CE 55 compactive effort. This repair should be constructed on a well compacted backfill material (dry density at least 85 percent CE 55 effort).

e. Cement stabilized gravel also served in a satisfactory manner with some deformation. It is suggested that 18 in. of cement stabilized gravel be placed in the crater to support anticipated loadings. The backfill below the stabilized gravel must also be compacted to a minimum of 85 percent CE 55 compactive effort.

Repair Responsibility

211. The engineer unit normally responsible for major port construction and rehabilitation is the engineer construction group. The group is organized to include specialized port construction units, construction battalions, dump truck companies, engineer pipeline companies, and other units as needed to accomplish the mission.

212. The engineer construction battalion (heavy) is the unit which will have the responsibility for conducting repairs within the port environment. It will be assisted in the area of waterfront construction by the engineer port construction company, which has the responsibility for pier/wharf repair. While the port construction company may be employed on container storage areas and other paved hardstands repair, these repair projects are more appropriately assigned to engineer construction battalions.

General Crater Repair Activities

213. Container storage area and other hardstand repair consists of preparing the crater bowl and the crater cap (Figure 76). The bowl consists only of backfilled debris and aggregate. This material provides foundational support for the cap. The cap consists of either one of the recommended repairs discussed in paragraph 210 (crushed stone, grouted aggregate, PCC, AC, or cement stabilized gravel). The cap provides strength, acts as a sealant against moisture, and restores a suitable smooth surface.

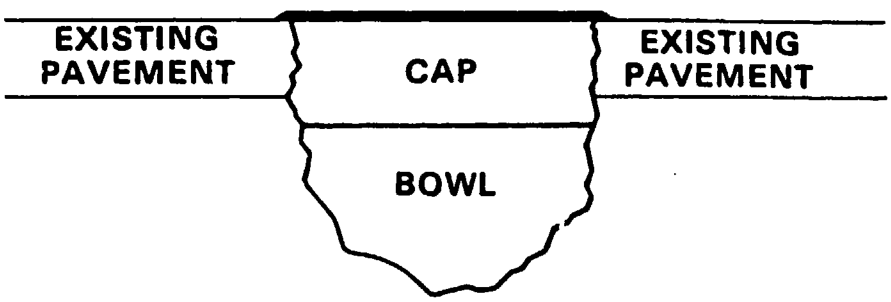


Figure 76. Crater bowl and cap

214. The bowl is prepared by removing all standing water from the crater bottom by the most efficient means available. Water removal is necessary to allow for proper compaction of subsequent fill layers and to avoid excessive settlement of the cap. Pneumatic pumps are ideally suited for water removal. If pumps are not available, troops using empty cans from the dining facility or buckets can manually remove the standing water.

215. Crater debris will be used in large volumes for crater backfill. Unsuitable debris must be disposed of in the early stages of the backfill operation. Therefore, the debris should be generally classified either as backfill debris or as waste debris. The backfill debris is that material that is necessary and suitable for use in filling craters. Pavement pieces larger than 12 in. and large amounts of clay or silt materials should not be placed as backfill debris. The waste debris is that material not needed or material that is unsuitable because of size, shape, or physical properties.

216. A 2-1/2- or 5-yd loader and a D7 bulldozer work as a team in backfilling the crater bowl with select debris. The loader pushes backfill debris into the crater and the bulldozer enters the crater to spread and compact the debris. The loader also pushes all large debris and unsuitable ejecta to the

side of the crater for site removal. After the crater bowl has been filled to an appropriate grade, the backfill is compacted to at least 85 percent CE 55 compactive effort and the aggregate to at least 95 percent CE 55 compactive effort.

217. After the crater bowl has been completed, a cap consisting of one of the recommended repairs discussed in paragraph 210 is selected and placed. Details of placement, personnel, and equipment are presented in field circular, FC 5-104-1 (US Army Engineer School 1985). Final operations consist of brooming surrounding areas for loose debris removal.

PART VI: CONCLUSIONS AND RECOMMENDATIONS

Conclusions

218. In a military conflict, hostile action can be anticipated against port facility targets to render the facilities inoperative or to deny access to these port facilities.

219. Port facilities damaged by military confrontations will impede the movement of containerized and other cargo from ship to shore and inland.

220. The port vulnerability analysis indicates that pier/wharf facilities closest to the water's edge should be of major concern and prime targets during military attacks.

221. The design loads for containerized cargo are greater than the loads of previous general cargo operations.

222. Many ports are not designed for the use of mobile cranes or container handling forklifts on the wharf. Containers are normally handled by rail mounted cranes at wharfside and then hauled to storage areas by either straddle carriers or tractor-trailers.

223. The strength of pier/wharf repairs should match the as-constructed strength of the wharf.

224. Pier/wharf deck repair/replacement with cast-in-place concrete is a time, material, and labor-consuming process.

225. Precast concrete pier/wharf components can be used as a construction process to expedite the cast-in-place concrete repair process. Cast-in-place construction is slowed by the continuous building and stripping of formwork.

226. Army port construction units lack sufficient training and equipment in concrete construction.

227. There are a number of pieces of military bridging equipment that can be used as expedient repairs to bridge war-damaged areas of piers/wharves. Military bridging is not normally designated as a port repair mission; however, if bridging were available it could be used to repair piers/wharves.

228. The truss web aluminum extrusions and anchorage system are potential rapid repair items for war-damaged pier/wharf decking. Based on results obtained in traffic tests, the following are warranted:

a. Twelve-foot-long truss web aluminum extrusions will sustain at least 1,000 passes each of a 43,900-lb single-wheel load and a 16,450-lb dual-wheel load when placed over craters up to 6 ft in diameter. This represents the loading for a Belotti straddle carrier and a M878 yard tractor/M872 trailer used for container handling.

b. Craters with diameters greater than 6 ft but not more than 8.4-ft in diameter when repaired with 24-ft-long extrusions will sustain at least 1,000 passes each of the above wheel loads.

c. Two anchor bolts per truss web extrusion end when placed in concrete are necessary to properly anchor the extrusions.

229. Repair techniques for airfield pavements are applicable to repair bomb-damaged storage areas and other facility pavements at ports.

Recommendations

230. US Army units should participate in any testing and evaluation of new port repair systems.

231. Simulated training missions should be performed on piers which are damaged or scheduled for demolition to allow military units to acquire actual experience.

232. Information presented in this report should be used to update TM 5-360, dated September 1964 (Headquarters, Department of the Army 1964).

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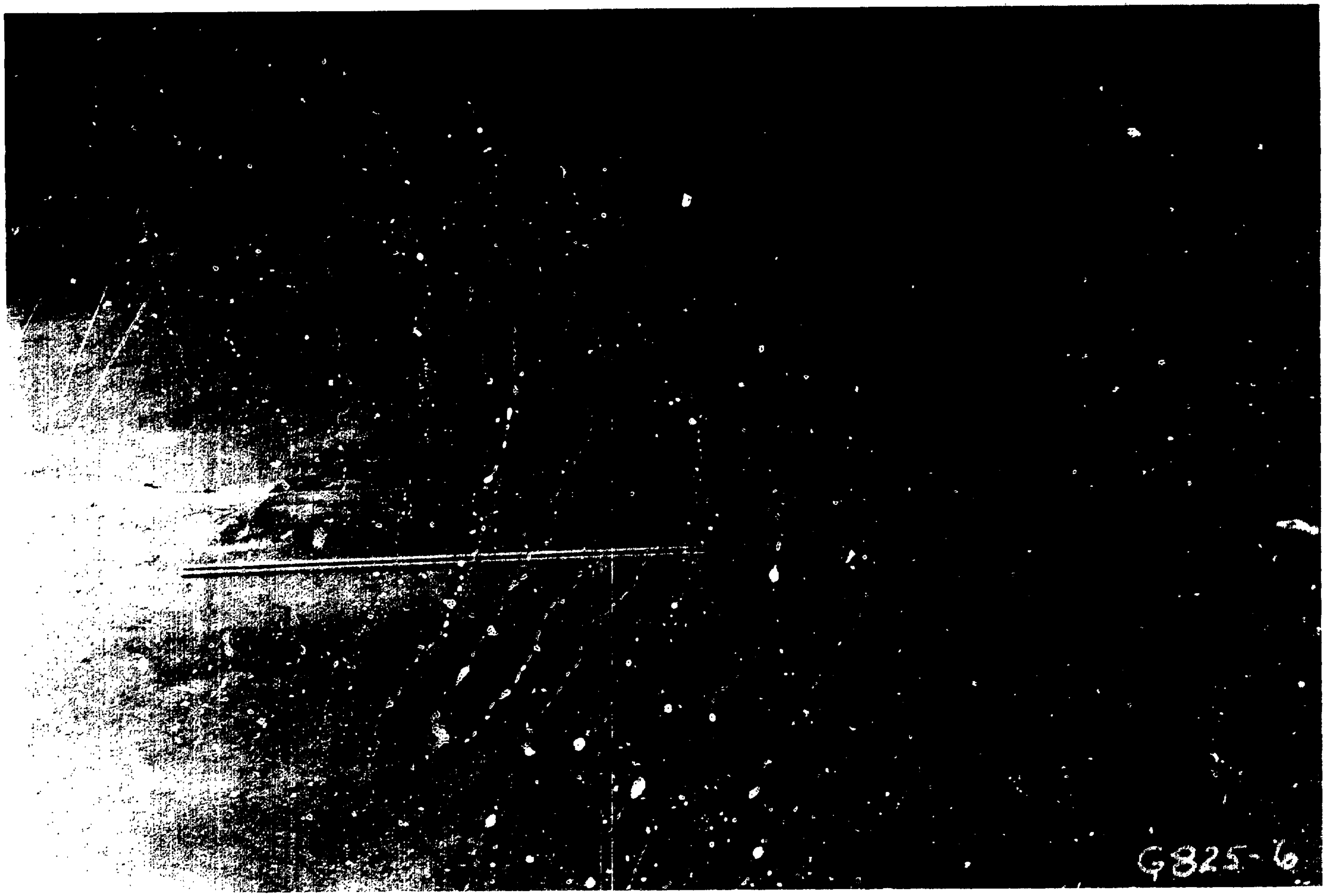


Photo 1. Test section prior to traffic, Test 1

Photo 2. Test section after 1,000 passes, Test 1

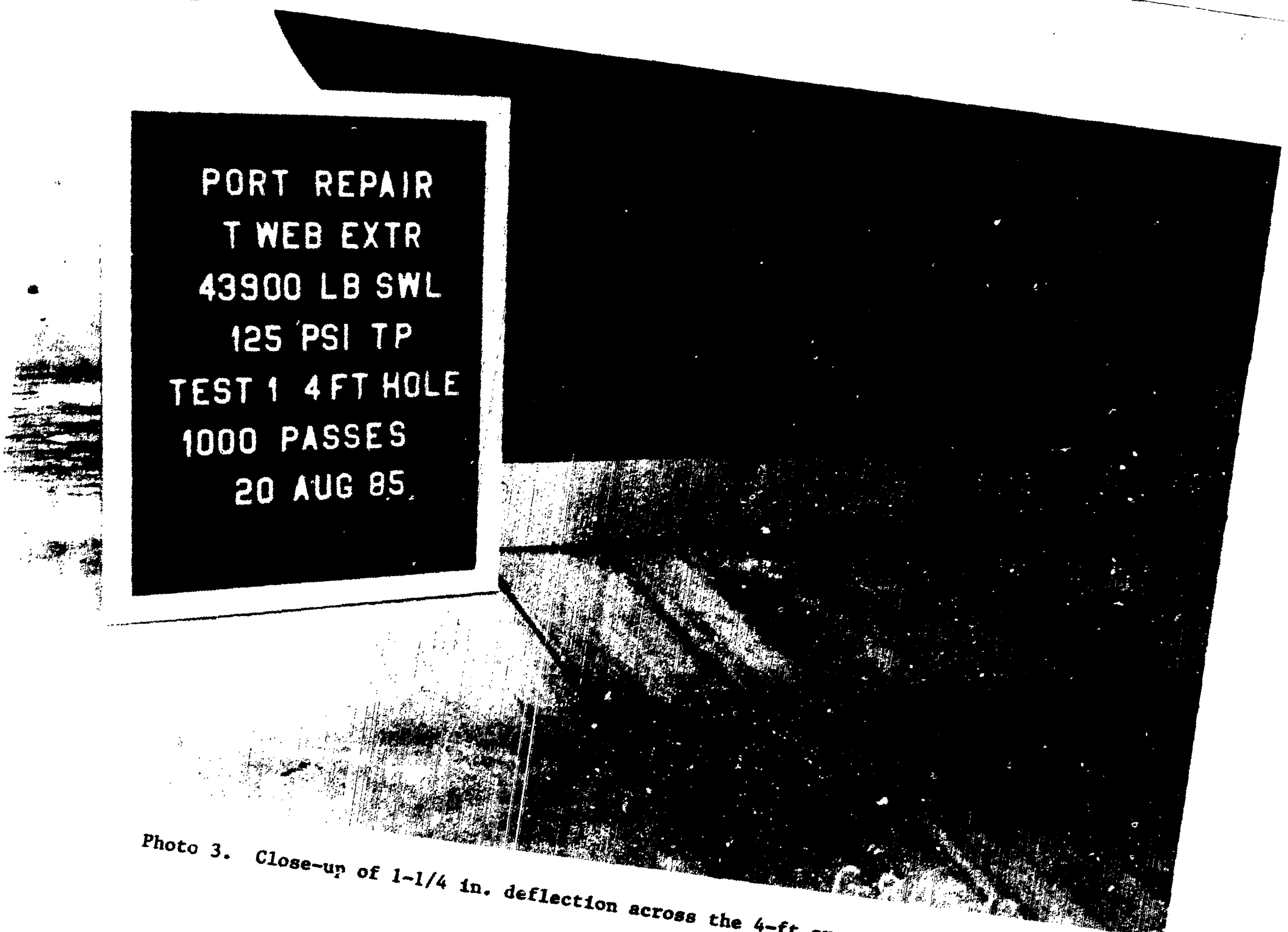


Photo 3. Close-up of 1-1/4 in. deflection across the 4-ft crater, Test 1

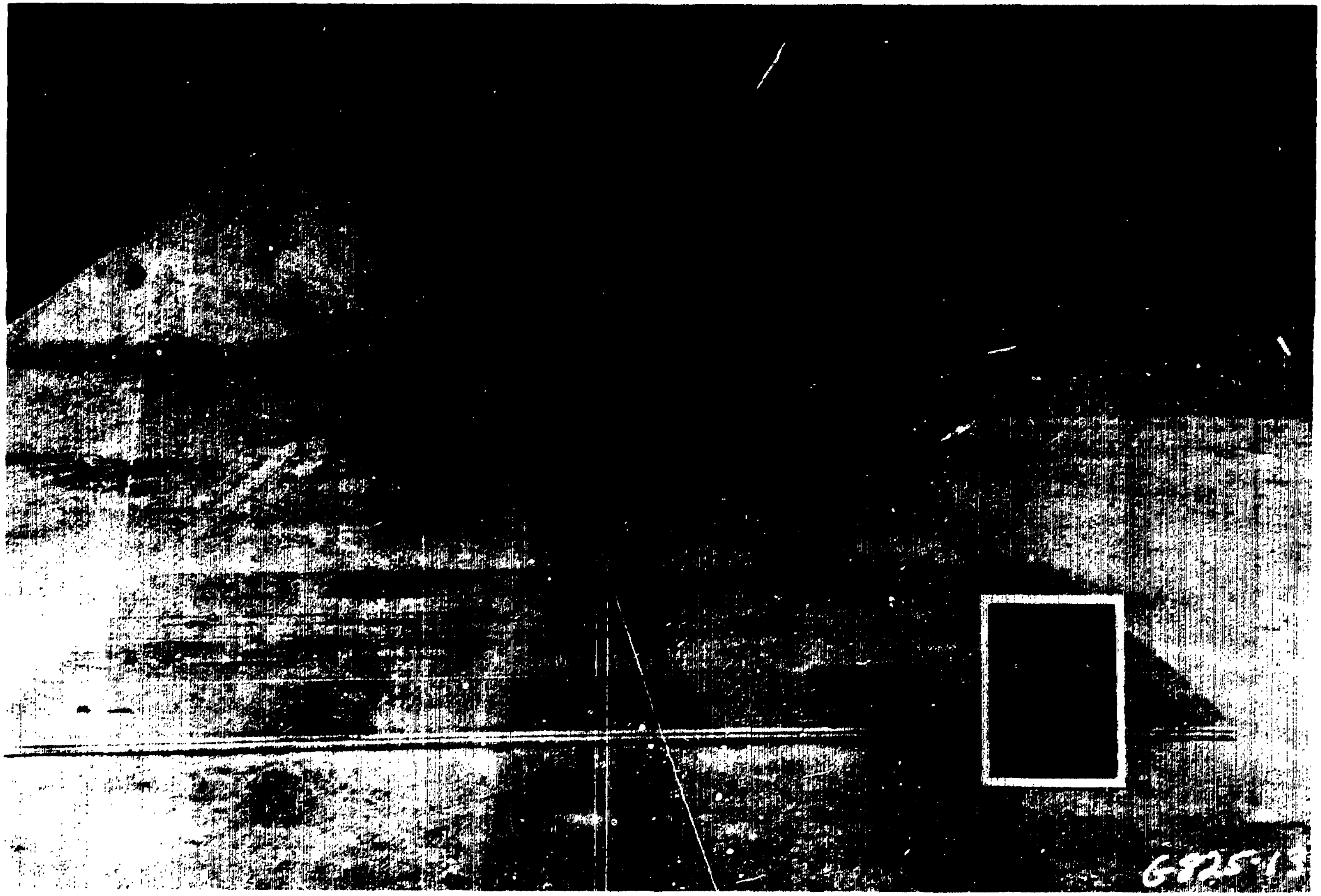


Photo 4. Test section prior to traffic, Test 2

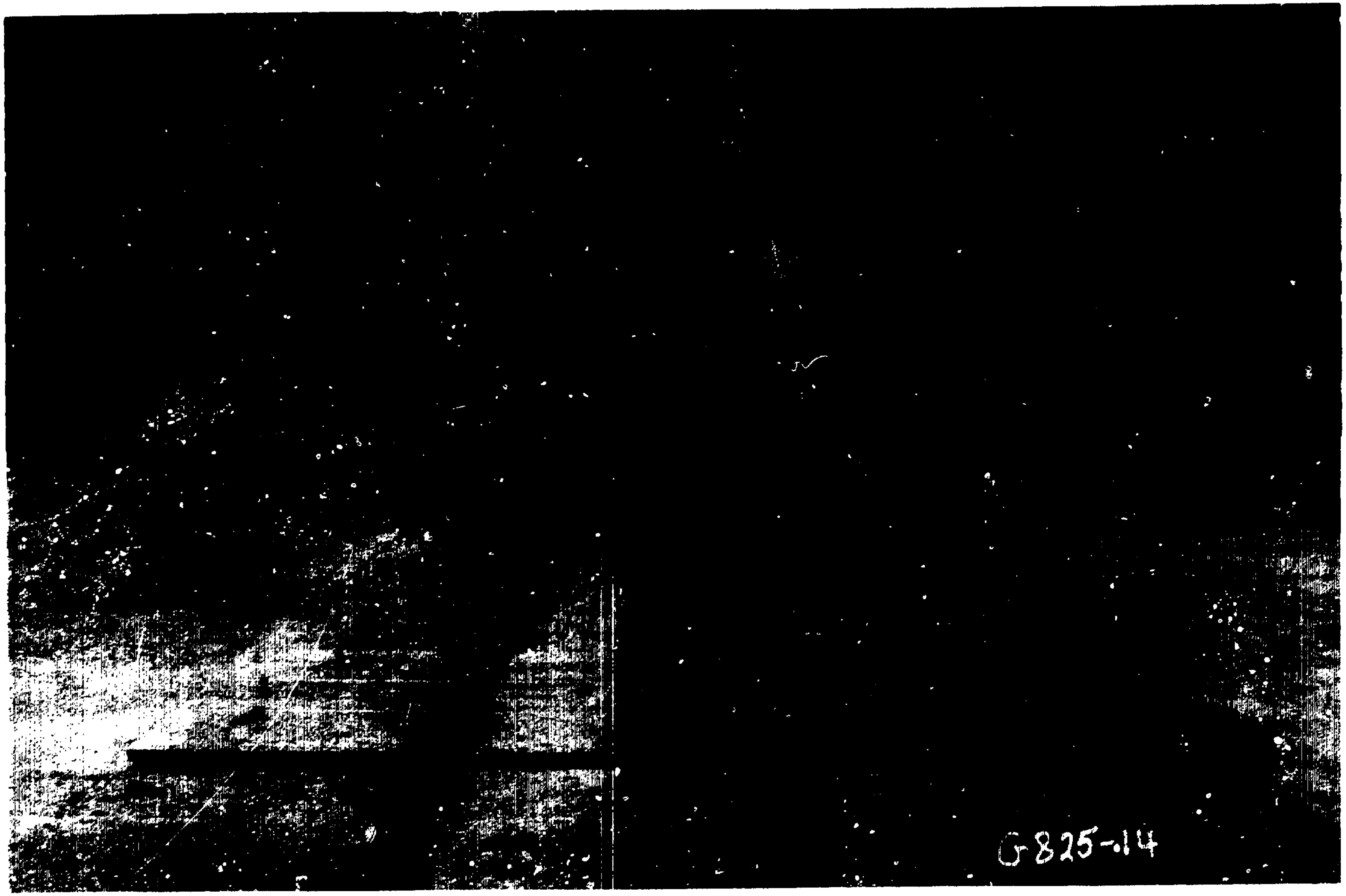


Photo 5. Test section after 72 passes with anchor bolts pulled upward

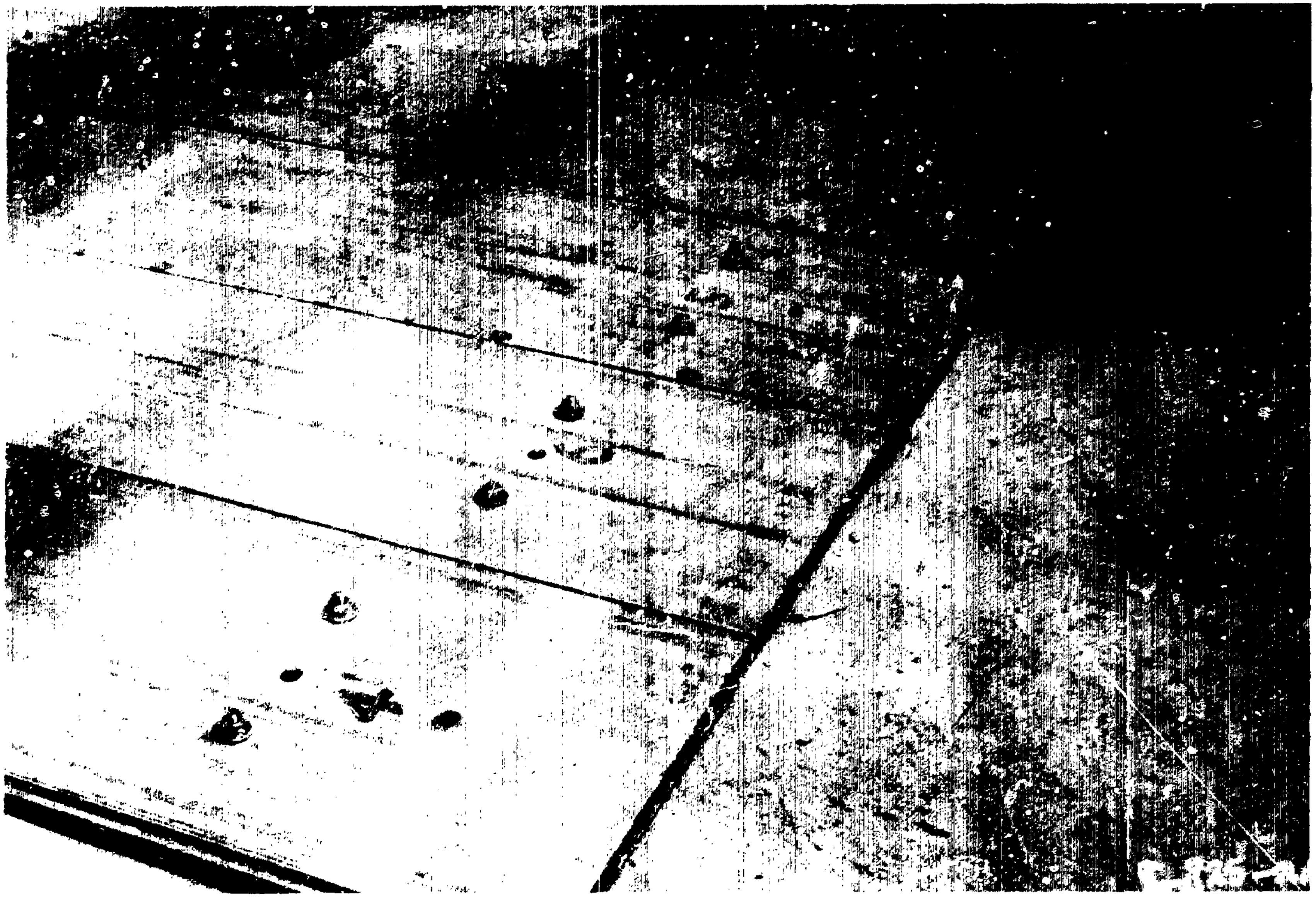


Photo 6. Two anchor bolts placed at each end of the extrusions

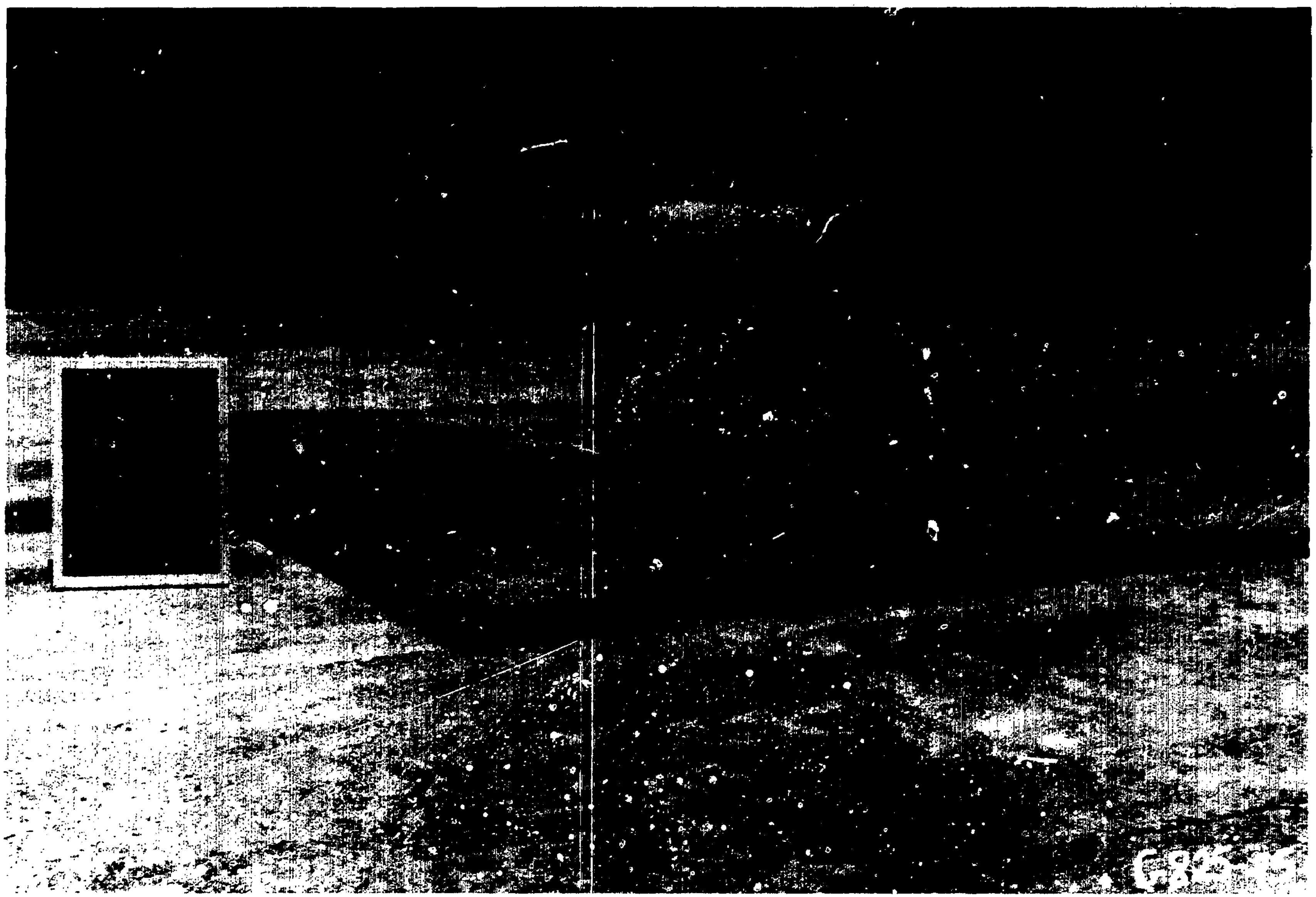


Photo 7. Anchor bolts pulled upward after 160 total passes

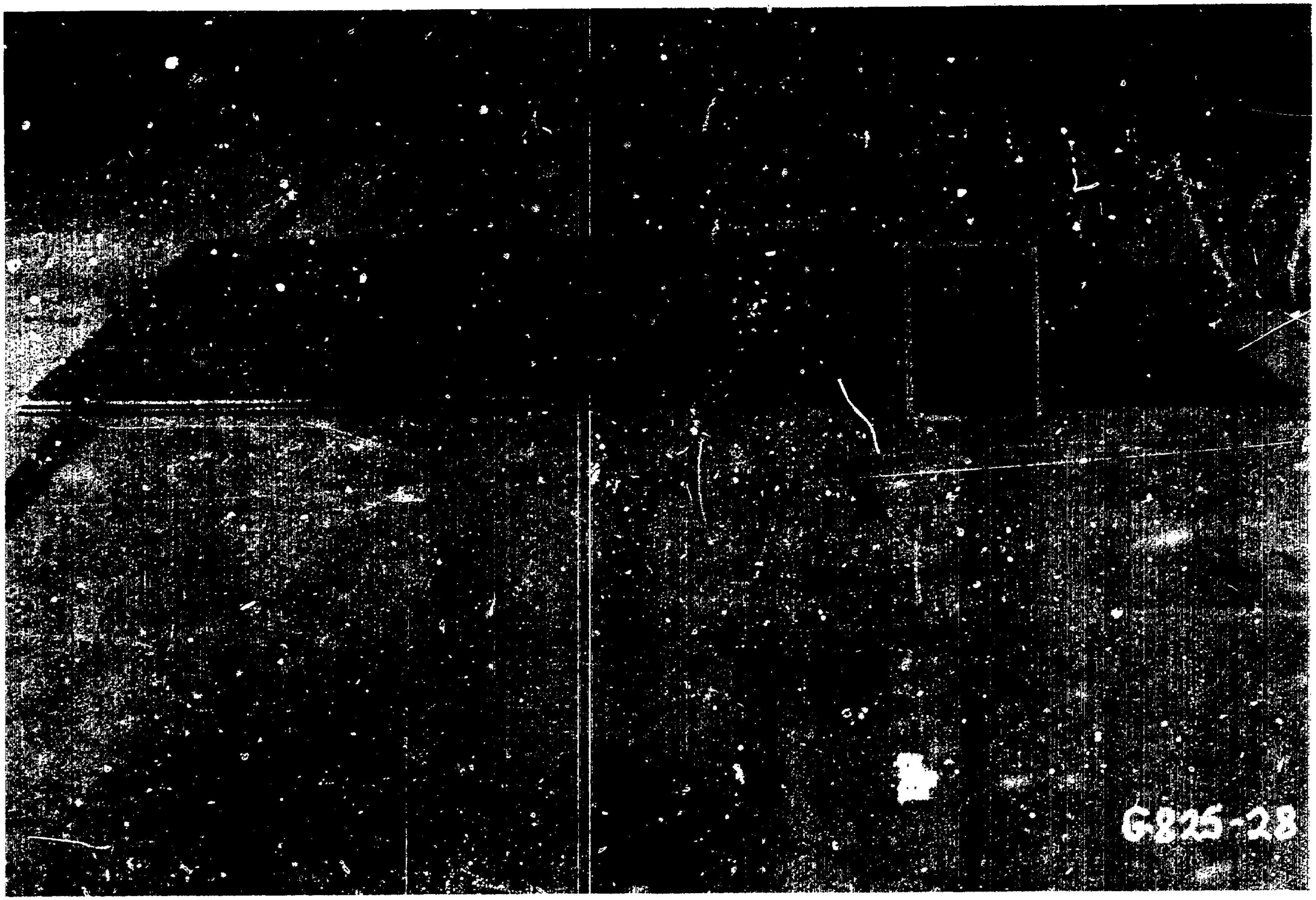


Photo 8. View of Test 2 section pric to traffic with extrusions anchored in concrete

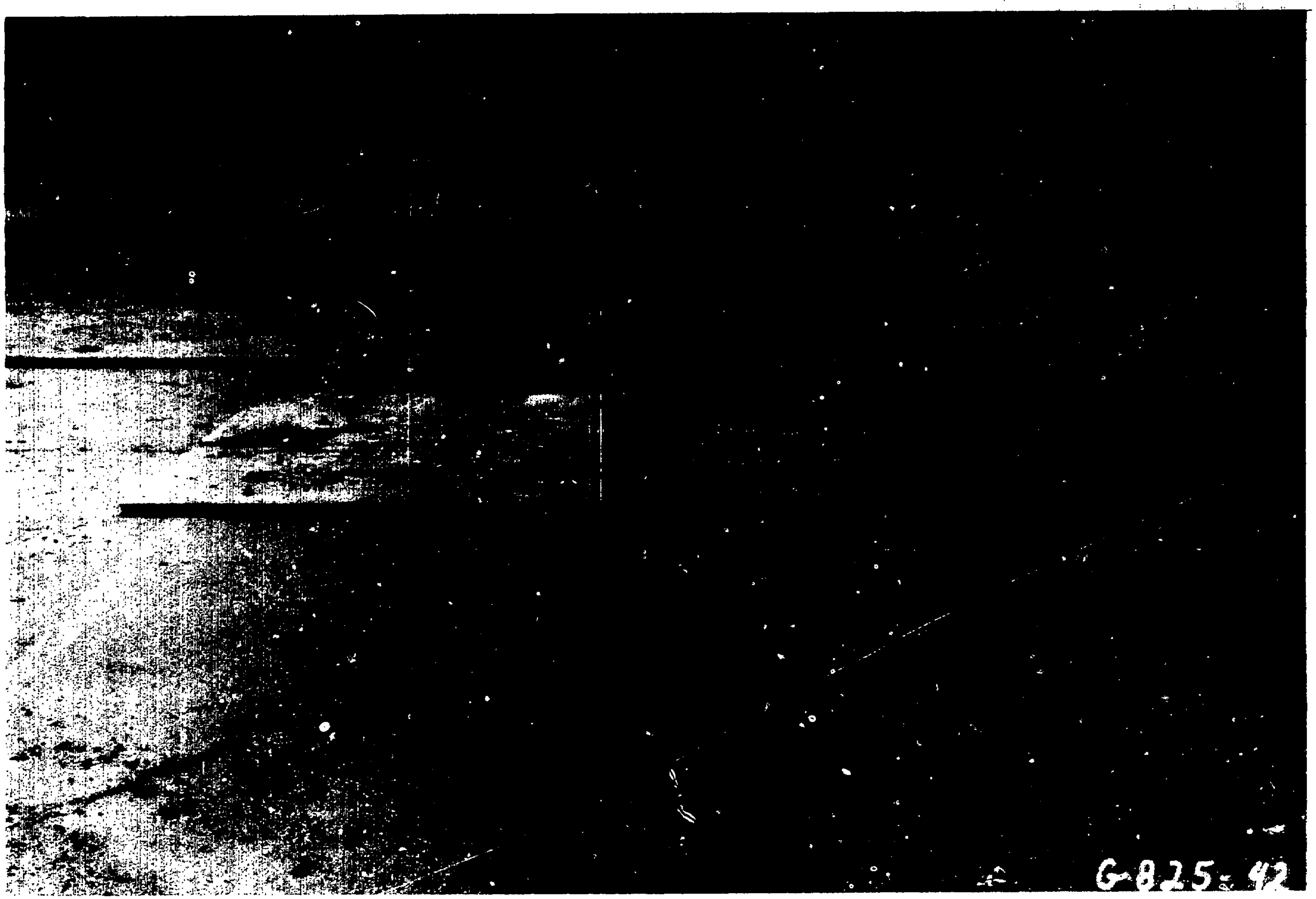


Photo 9. Test section after 1,000 passes, Test 2

Photo 10. Close-up of 1-5/8 in. deflection across the 6-ft crater, Test 2

Photo 11. Test section prior to traffic, Test 3

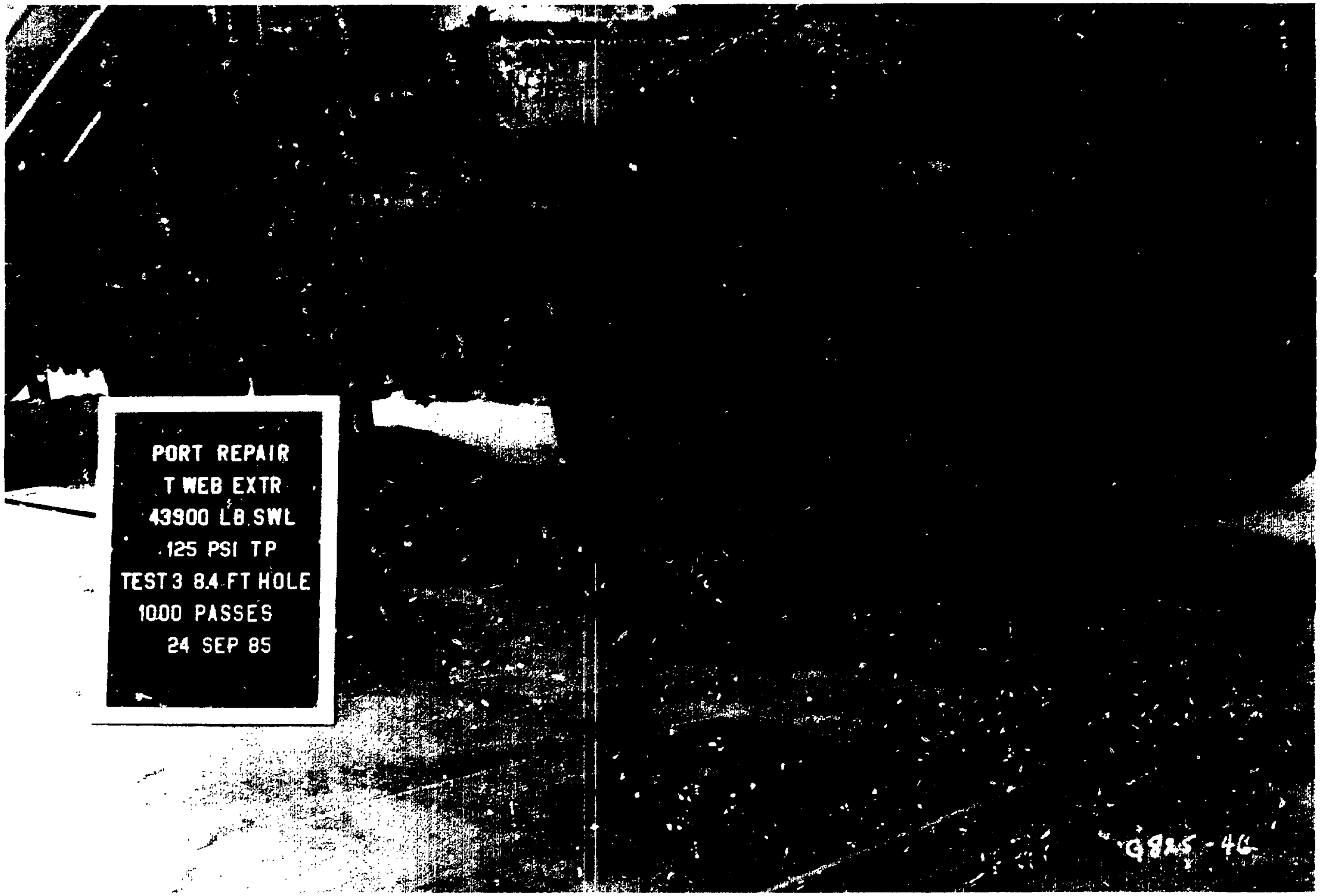


Photo 12. Close-up of 3-15/16 in. deflection across the 8.4-ft crater, Test 3

Photo 13. Close-up of 7/8 in. permanent set across the 8.4-ft crater after 1,000 passes

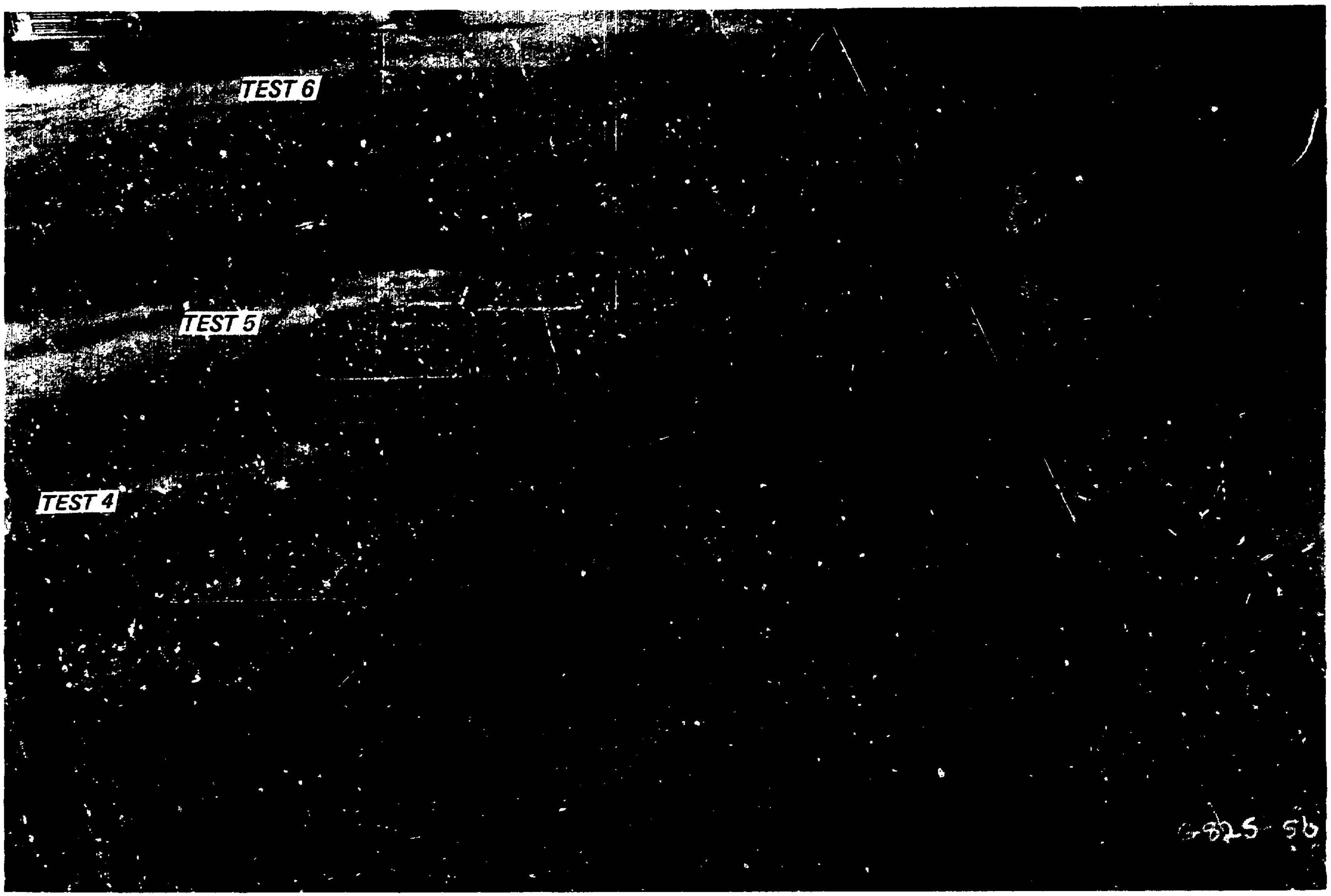


Photo 14. Overall view of test section prior to traffic

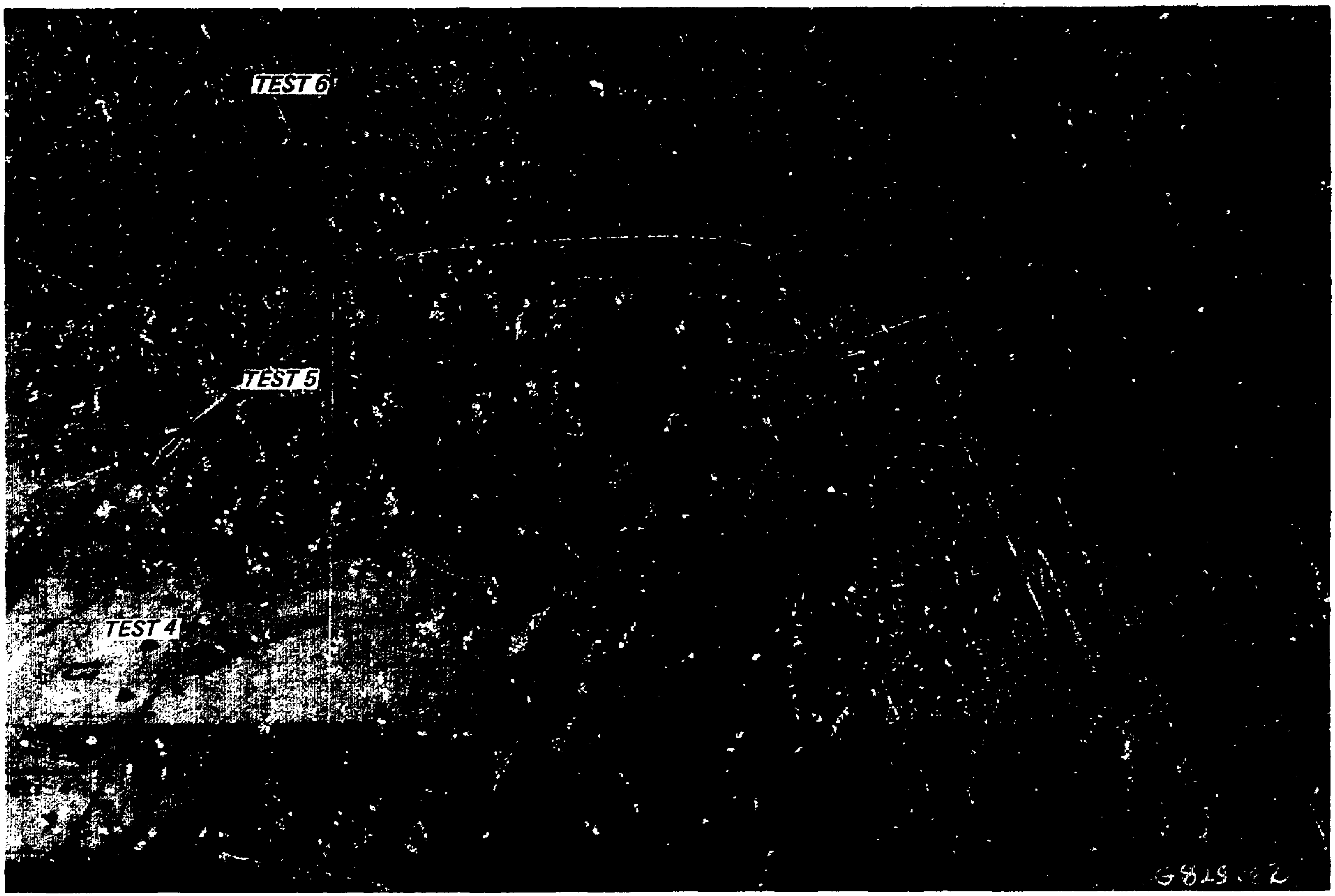


Photo 15. General view of test section after 1,000 passes

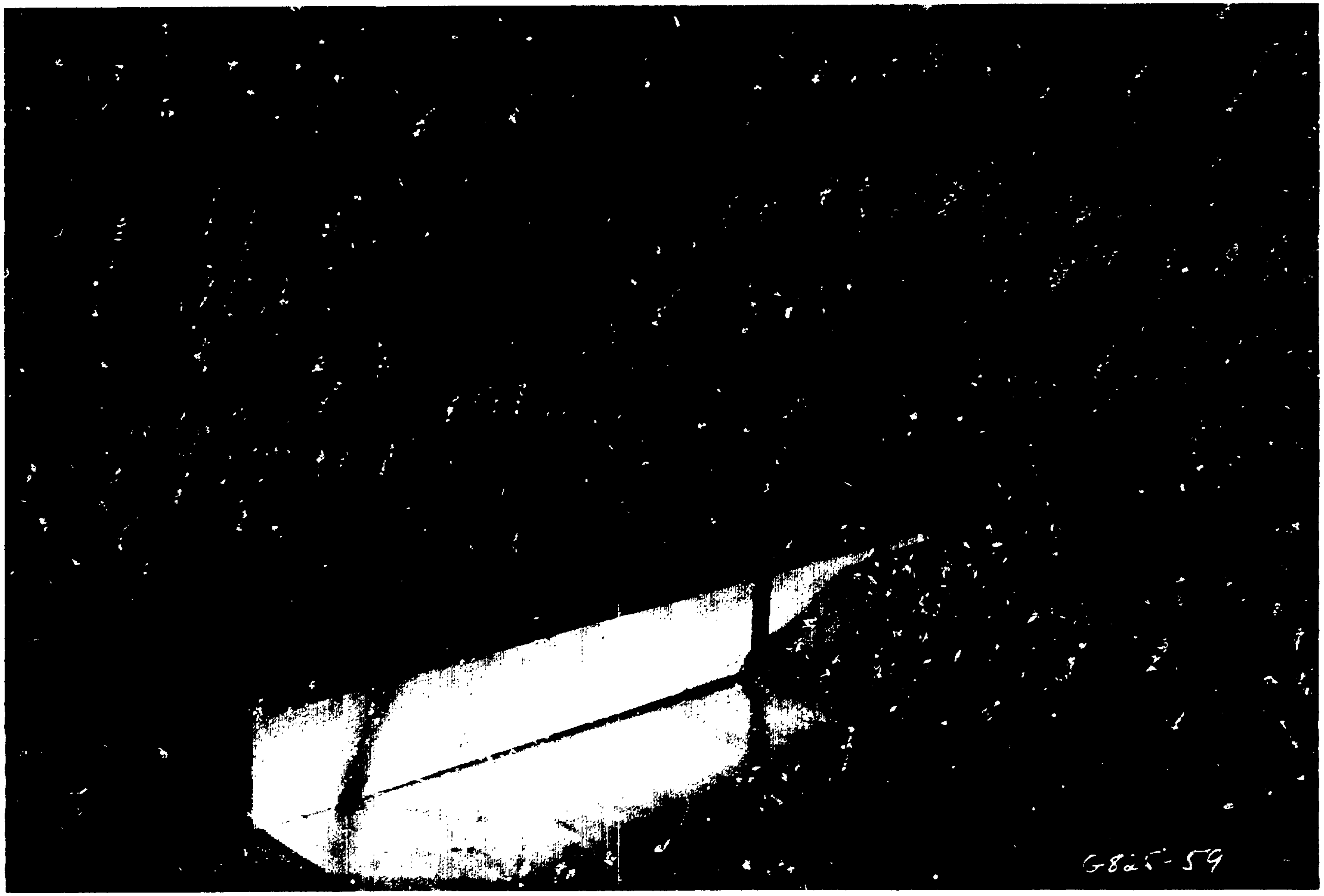


Photo 16. Close-up of 1/4 in. deflection across 4-ft crater (Test 4) after 1,000 passes

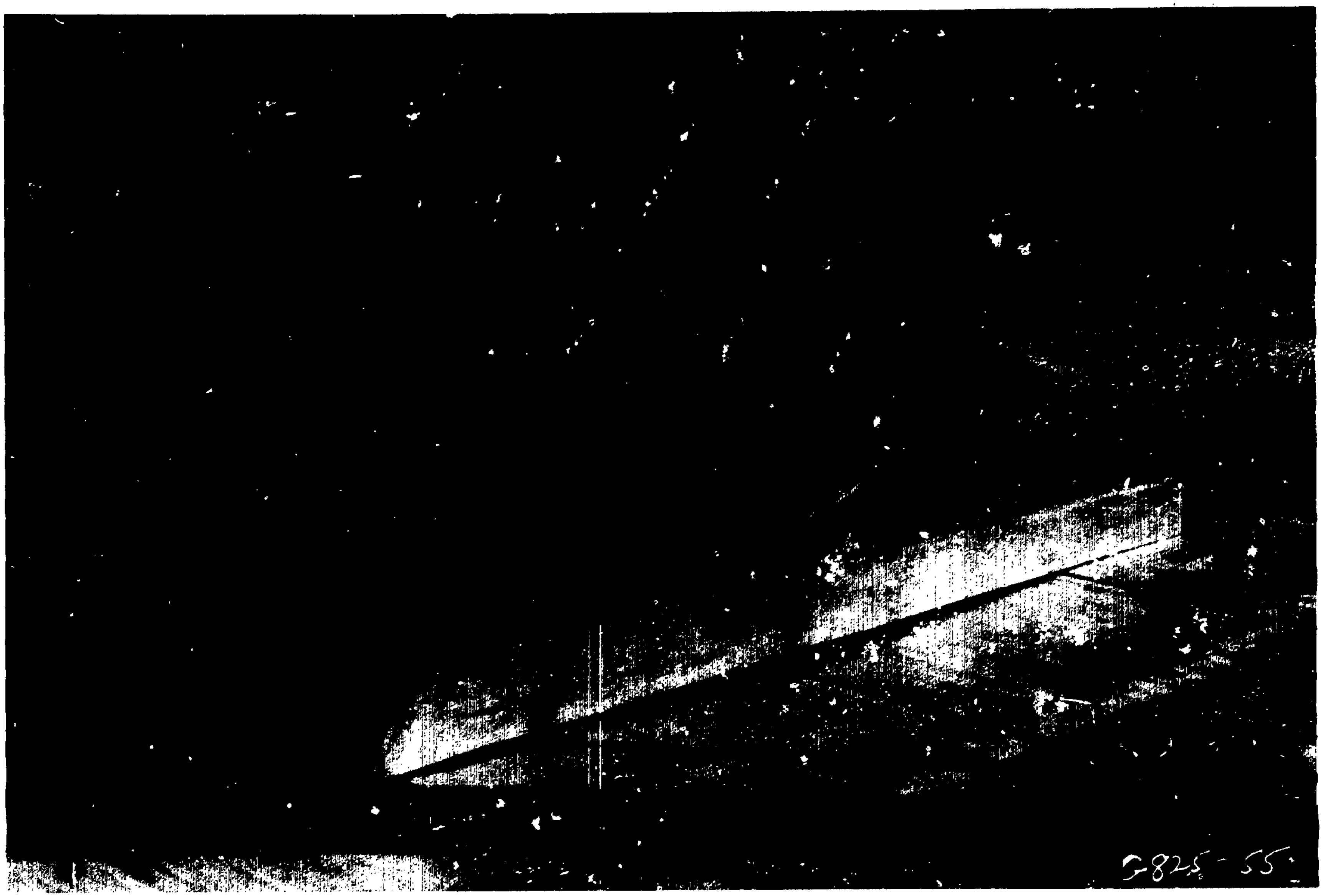
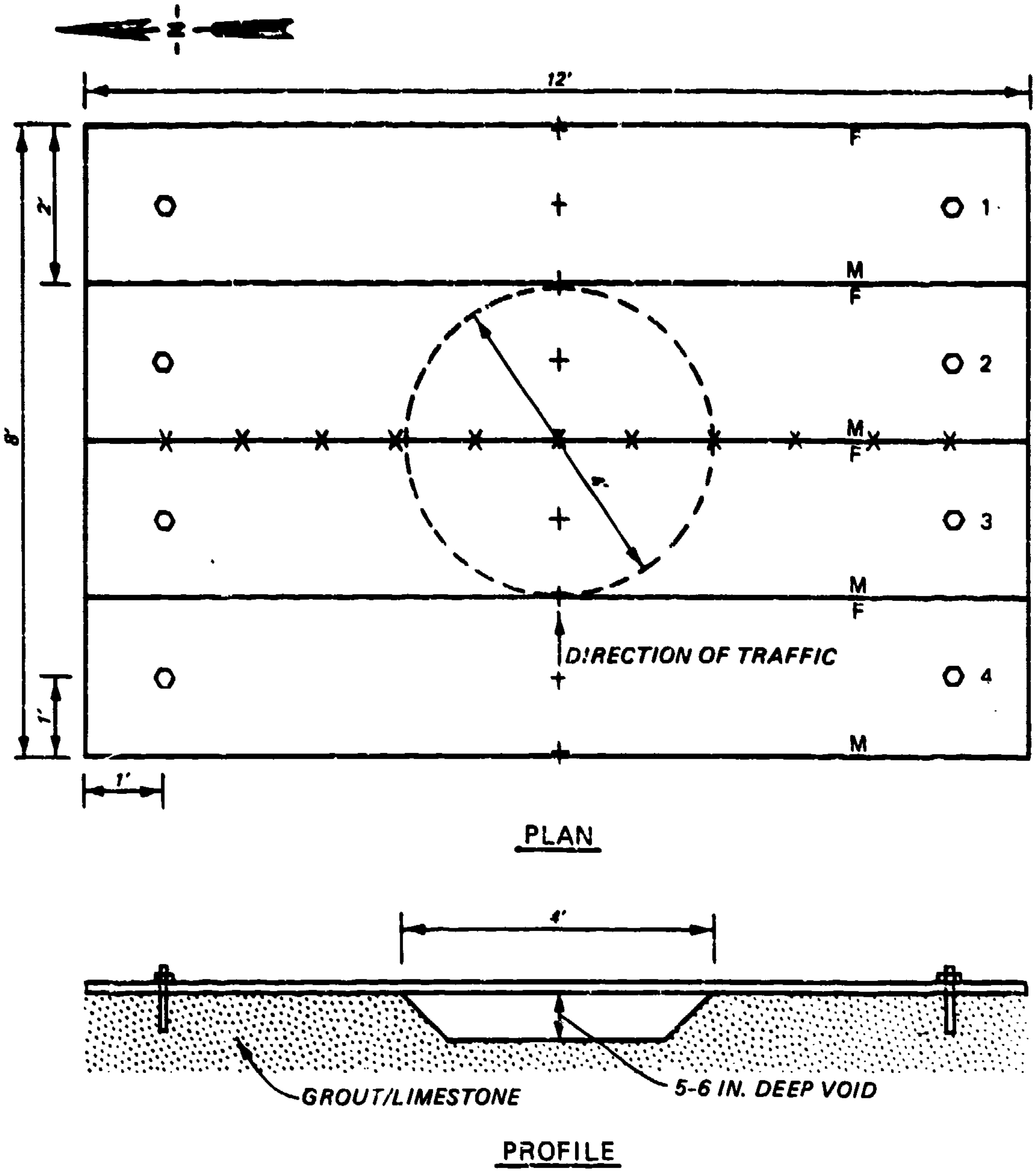


Photo 17. Close-up of 7/8 in. deflection across 6-ft crater (Test 5) after 1,000 passes

Photo 18. Close-up of 2-1/2 in. deflection across 8.4-ft crater (Test 6) after 1,000 passes



LEGEND

O – ANCHOR BOLTS

M – MALE CONNECTOR

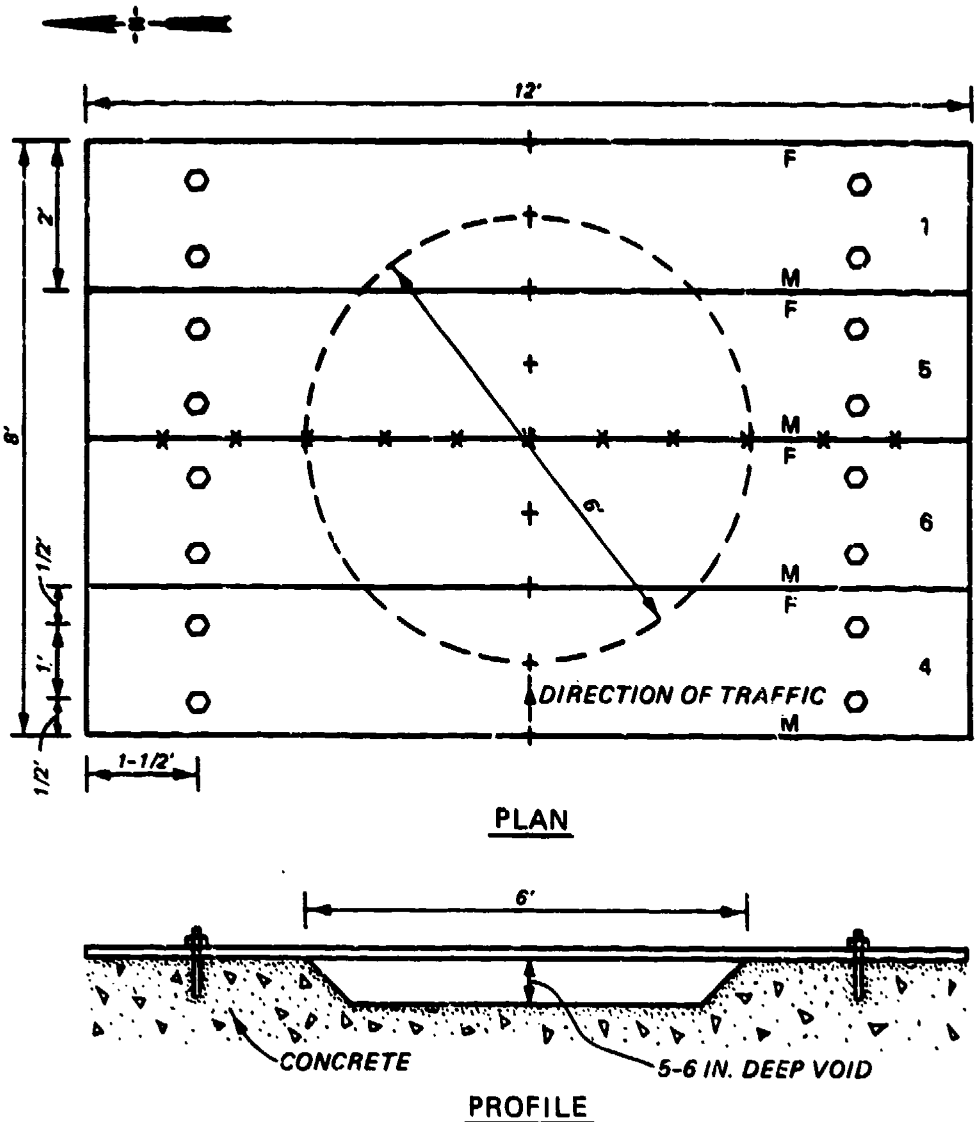
F – FEMALE CONNECTOR

X – CROSS SECTION MEASUREMENTS

+ – PROFILE MEASUREMENTS

NOTE: NUMBERS INSIDE PANELS INDICATE PANEL NUMBERS

LAYOUT FOR TEST 1
4-FT DIAMETER HOLE

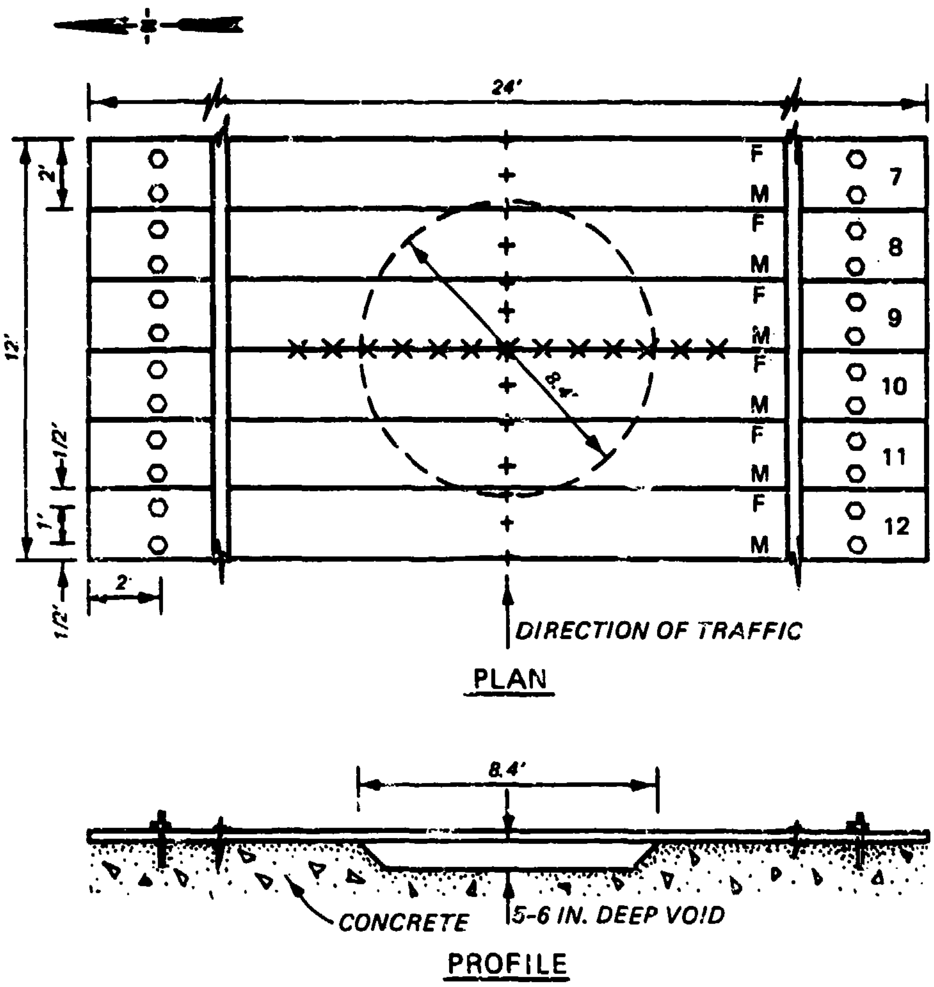


LEGEND

O – ANCHOR BOLTS
M – MALE CONNECTOR
F – FEMALE CONNECTOR
X – CROSS SECTION MEASUREMENTS
\+ – PROFILE MEASUREMENTS

NOTE: NUMBERS INSIDE PANELS INDICATE PANEL NUMBERS

LAYOUT FOR TEST 2
6-FT DIAMETER HOLE

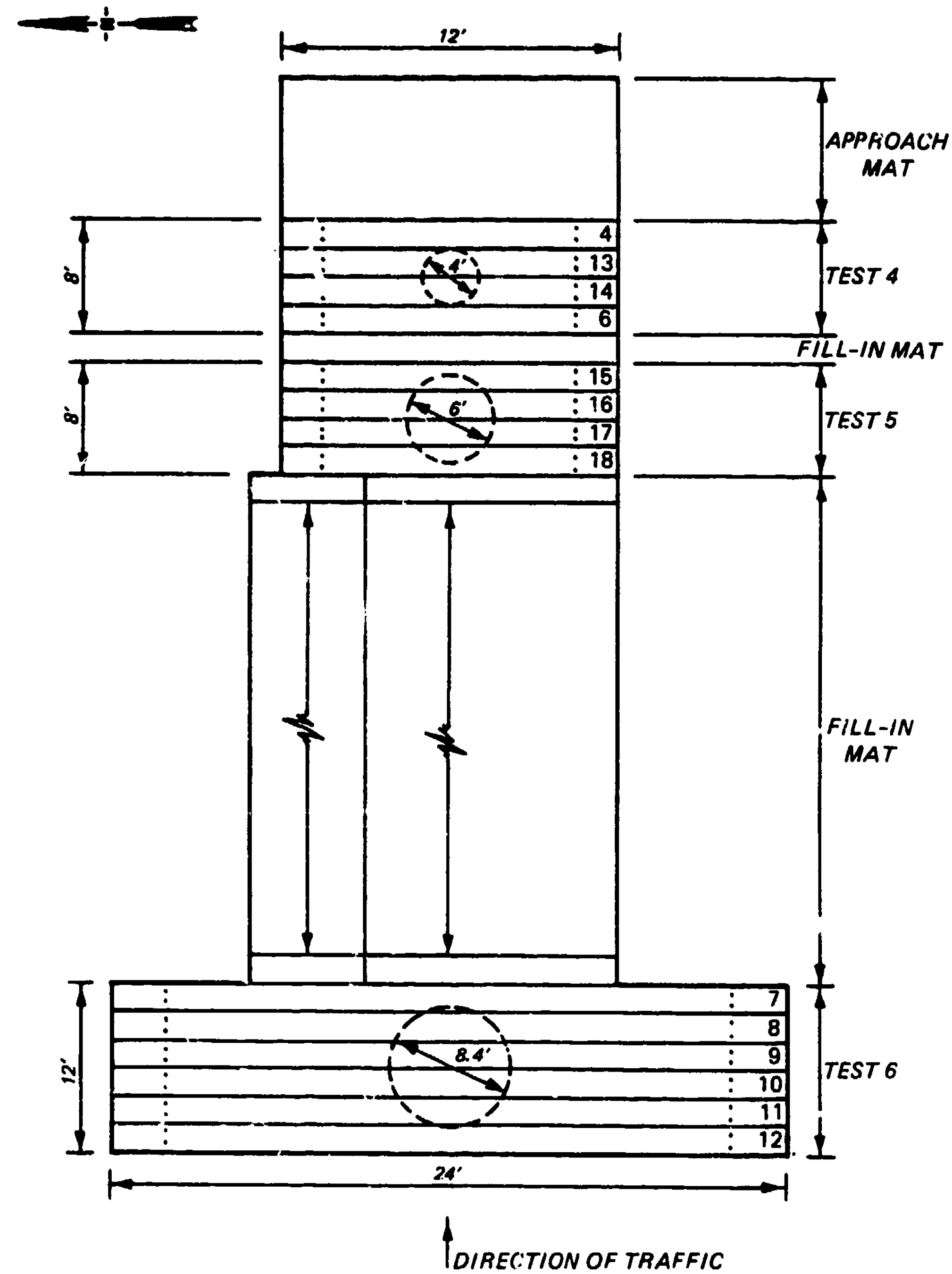


LEGEND

- O – ANCHOR BOLTS
- M – MALE CONNECTOR
- F – FEMALE CONNECTOR
- X – CROSS SECTION MEASUREMENTS
- \+ – PROFILE MEASUREMENTS

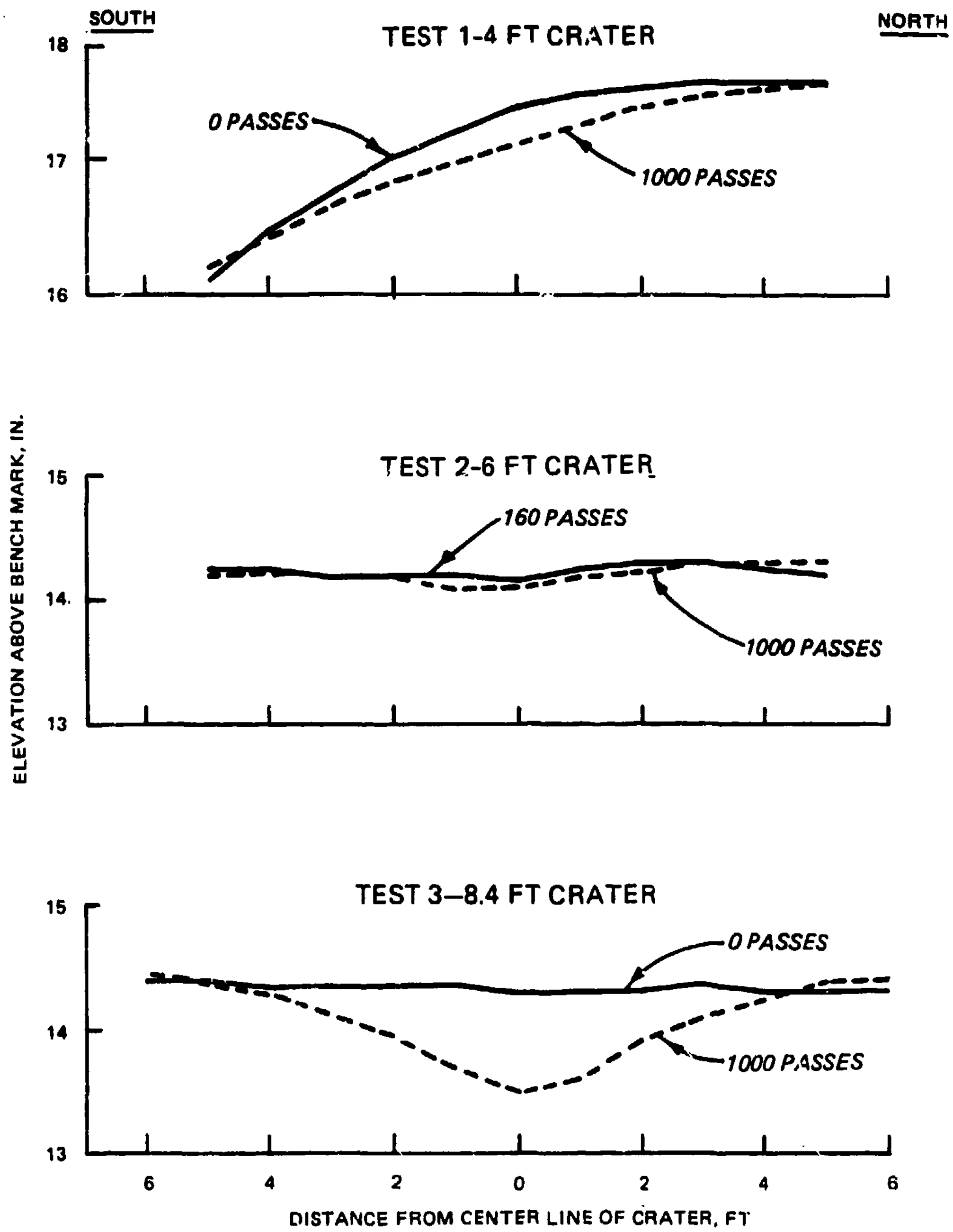
NOTE: NUMBERS INSIDE PANELS INDICATE PANEL NUMBERS

LAYOUT FOR TEST 3
8.4-FT DIAMETER HOLE

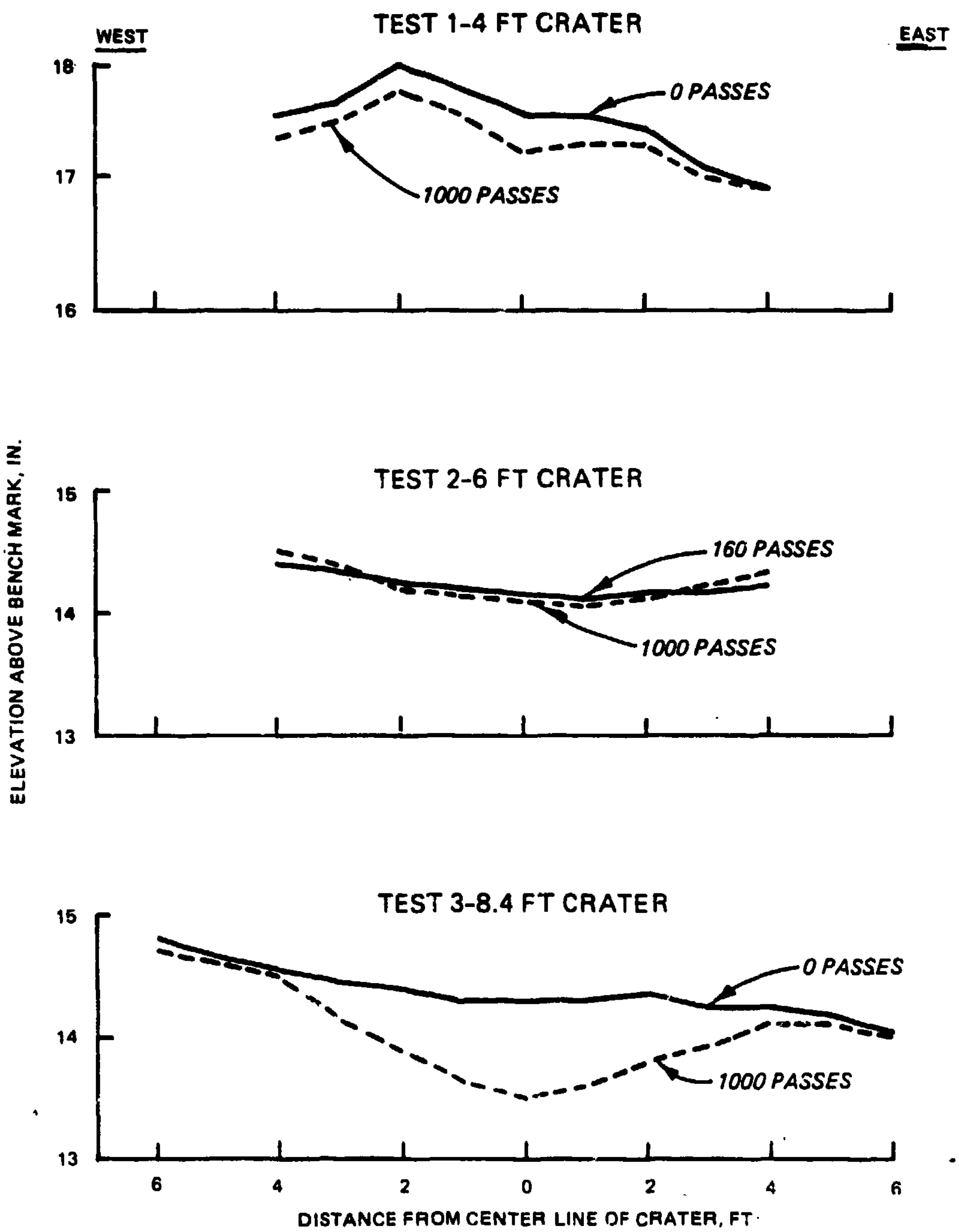


NOTE: NUMBERS INSIDE PANELS INDICATE PANEL NUMBERS

LAYOUT FOR TEST 4,5, AND 6
4-, 6-, AND 8.4-FT DIAMETER HOLES

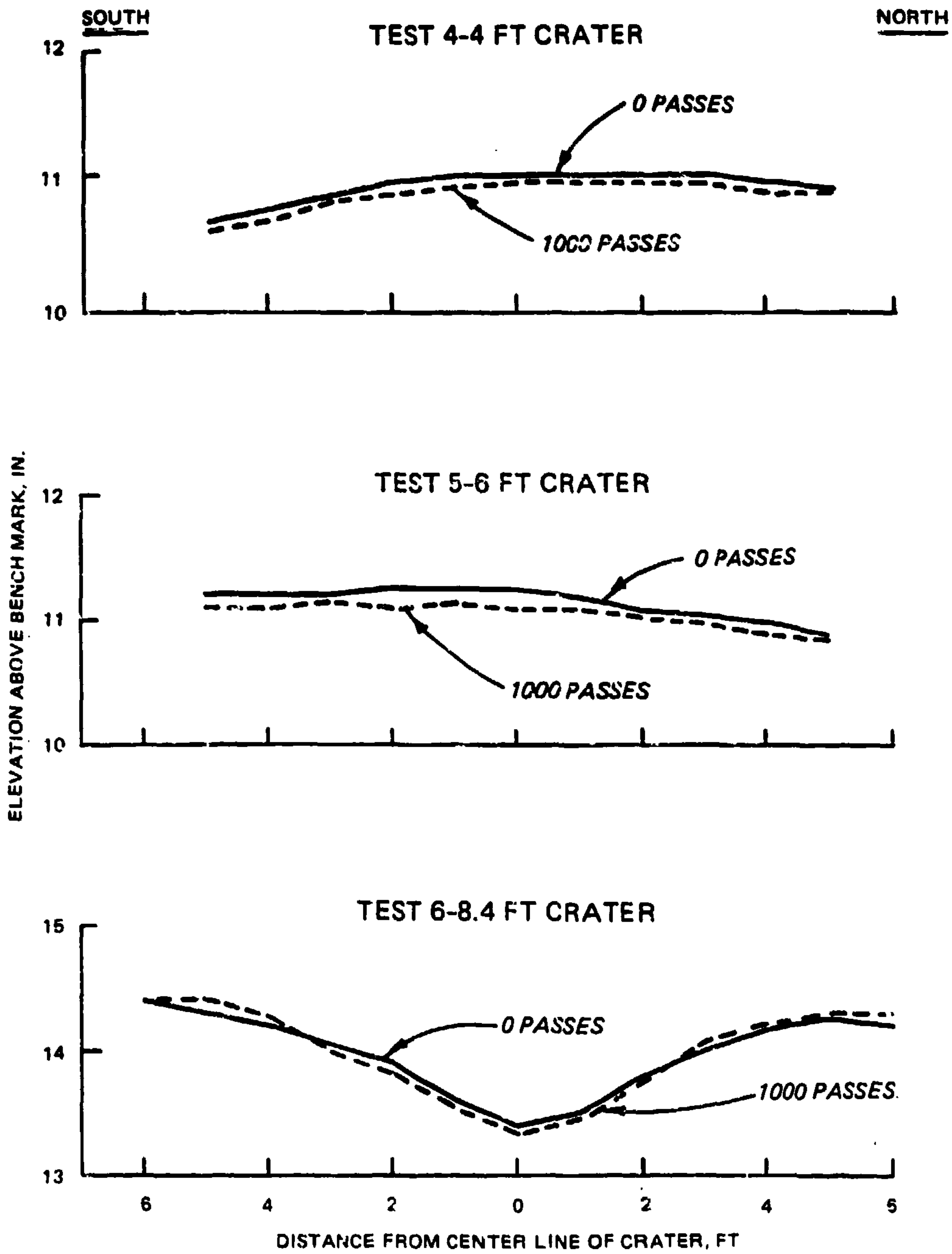


CROSS SECTIONS OF EXTRUSIONS

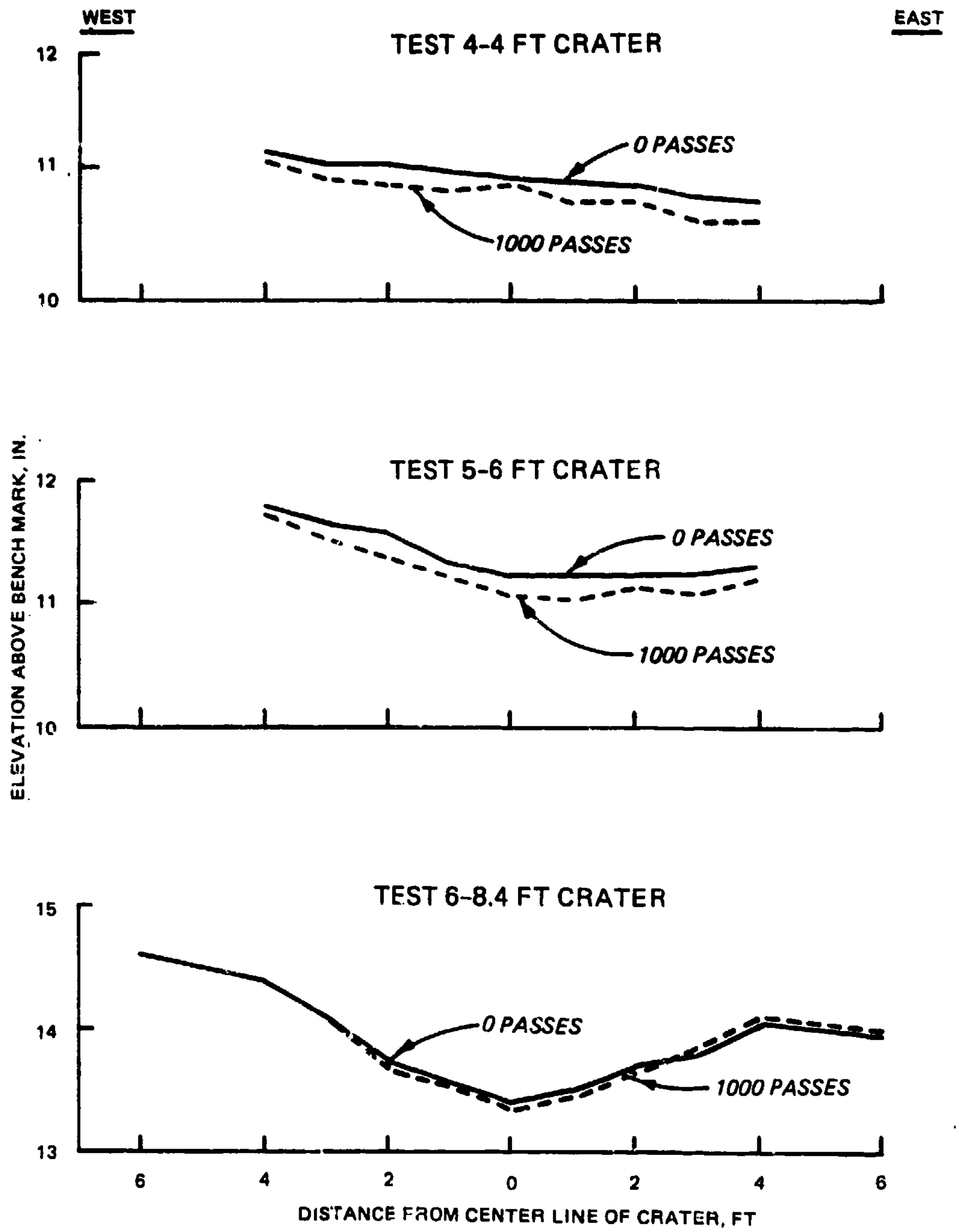


PROFILES OF EXTRUSIONS

PLATE 6



CROSS SECTIONS OF EXTRUSIONS

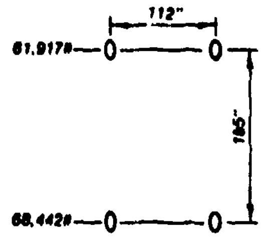


PROFILES OF EXTRUSIONS

PLATE 8

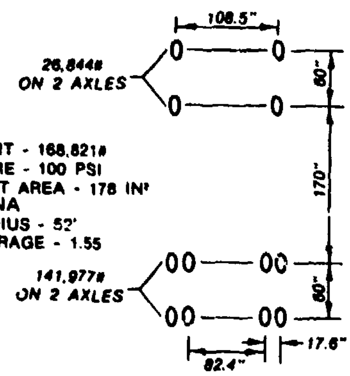
APPENDIX A: CHARACTERISTICS OF MATERIAL HANDLING VEHICLES

1. The following is a list of material handling vehicles with their characteristics. These vehicles will be used in port environments to move cargo and containers from pierside and in storage areas.



GROSS WEIGHT - 130,359#
TIRE PRESSURE - 55 PSI
TIRE CONTACT AREA - 688 IN²
PAYLOAD - NA
TURNING RADIUS - 30'
PASSES/COVERAGE - 1.39

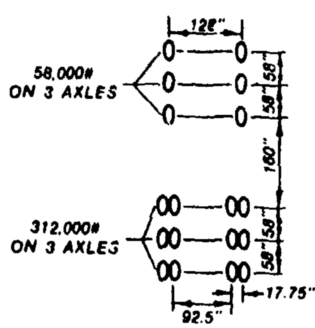
80-TON CRANE



GROSS WEIGHT - 168,821#
TIRE PRESSURE - 100 PSI
TIRE CONTACT AREA - 178 IN²
PAYLOAD - NA
TURNING RADIUS - 52'
PASSES/COVERAGE - 1.55

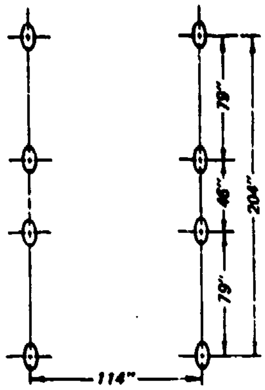
140-TON CRANE

NOTES:
1. GROSS WEIGHT WITH COUNTERWEIGHTS (370,000#)
2. GROSS WEIGHT LESS COUNTERWEIGHT 1 (319,000#)
3. GROSS WEIGHT LESS COUNTERWEIGHTS 1 AND 2 (288,000#)
4. GROSS WEIGHT LESS COUNTERWEIGHTS 1, 2, AND 3 (258,000#)
5. GROSS WEIGHTS DO NOT INCLUDE PAYLOADS



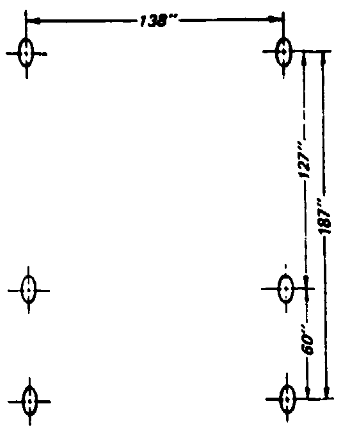
GROSS WEIGHT - 370,000#
TIRE PRESSURE - 100 PSI
TIRE CONTACT AREA - 260 IN²
PAYLOAD - NA
TURNING RADIUS - 59'
PASSES/COVERAGE - 0.60

250-TON CRANE



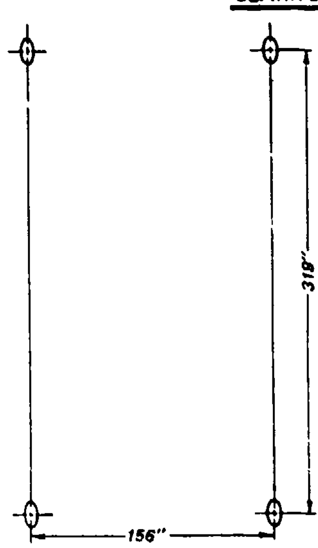
GROSS WEIGHT	=	129,200 LB
SINGLE-WHEEL LOAD	=	16,150 LB
TIRE INFLATION PRESSURE	=	100 PSI
CONTACT AREA	=	154 IN.^{2}
PAYLOAD	=	67,200 LB

SHOREMASTER STRADDLE CARRIER



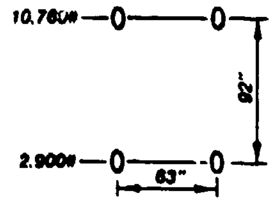
GROSS WEIGHT	=	164,500 LB
SINGLE-WHEEL LOAD	=	27,900 LB
TIRE INFLATION PRESSURE	=	132 PSI
CONTACT AREA	=	210 IN.^{2}
PAYLOAD	=	67,200 LB

CLARK 512 STRADDLE CARRIER



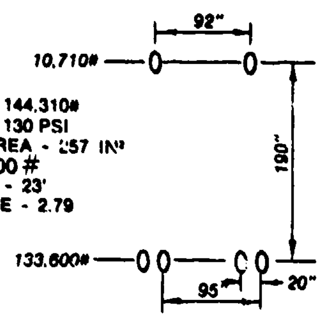
GROSS WEIGHT	=	159,800 LB
SINGLE-WHEEL LOAD	=	43,900 LB
TIRE INFLATION PRESSURE	=	125 PSI
CONTACT AREA	=	380 IN.^{2}
PAYLOAD	=	67,200 LB

BELOTTI B67b STRADDLE CARRIER



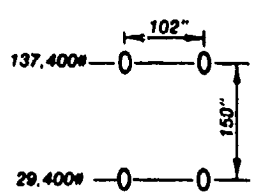
GROSS WEIGHT - 13,680#
TIRE PRESSURE - 45 PSI
TIRE CONTACT AREA - 120 IN²
PAYLOAD - 4,000 #
TURNING RADIUS - 11'
PASSES/COVERAGE - 4.8

4,000-LB FORKLIFT



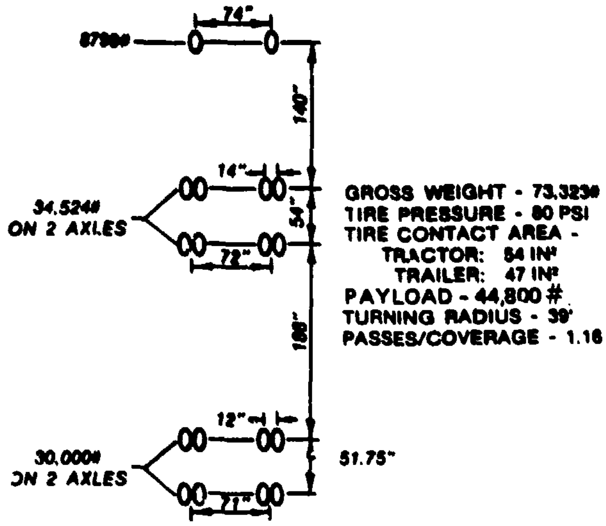
GROSS WEIGHT - 144,310#
TIRE PRESSURE - 130 PSI
TIRE CONTACT AREA - 257 IN²
PAYLOAD - 50,000 #
TURNING RADIUS - 23'
PASSES/COVERAGE - 2.79

HYSTER 620B FORKLIFT

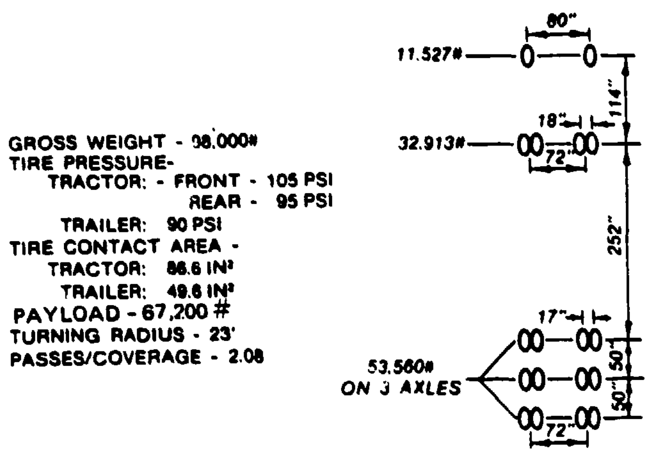


GROSS WEIGHT - 166,800#
TIRE PRESSURE - 92 PSI
TIRE CONTACT AREA - 747 IN²
PAYLOAD - 50,000 #
TURNING RADIUS - 35'
PASSES/COVERAGE - 2.72

CATERPILLAR 988B FORKLIFT

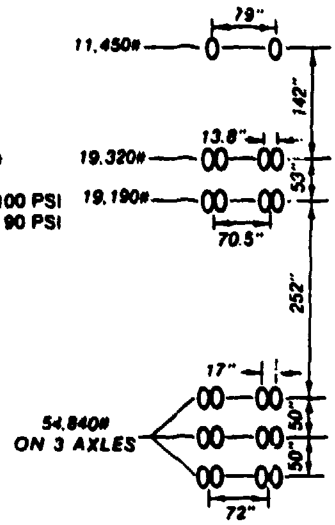


M52 TRACTOR WITH XM871 TRAILER

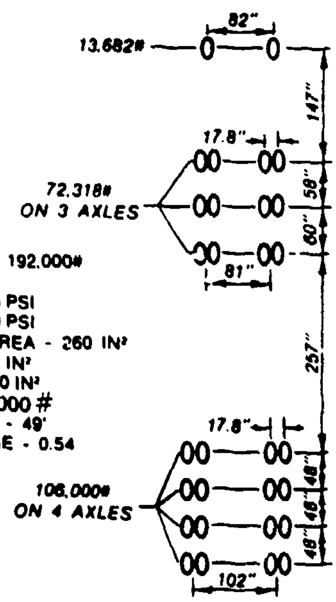


XM873 TRACTOR WITH XM872 TRAILER

GROSS WEIGHT - 104,800#
TIRE PRESSURE-
TRACTOR: - FRONT - 100 PSI
REAR - 90 PSI
TRAILER: 90 PSI
TIRE CONTACT AREA–
TRACTOR: 53.5 IN^2
TRAILER: 50.8 IN^2
PAYLOAD - 87,200 #
TURNING RADIUS - 42'
PASSES/COVERAGE - 0.94



M915 TRACTOR WITH XM872 TRAILER



GROSS WEIGHT - 192,000#
TIRE PRESSURE-
TRACTOR: 85 PSI
TRAILER: 60 PSI
TIRE CONTACT AREA - 260 IN^2
TRACTOR: 77 IN^2
TRAILER: 110 IN^2
PAYLOAD - 120,000 #
TURNING RADIUS - 49'
PASSES/COVERAGE - 0.54

M911 HEAVY EQUIPMENT TRANSPORTER

APPENDIX B: DECK REPLACEMENT WITH CAST-IN-PLACE CONCRETE

1. Timber structures damaged by bomb blast normally consist of areas which are either completely destroyed, partially destroyed, or not damaged at all. If the deck structure is so badly damaged that there are no useable areas, such as areas severely damaged by fire, then the deck must be removed to the substructure level and formwork constructed to cast a new deck. This is a time, material, and labor-consuming process but one that allows the engineer to choose the capacity of the new decking to correspond with mission needs. If the timber deck is not entirely destroyed, the deck can be repaired as well as strengthened by placing a concrete overlay above the good quality timber decking.

2. The procedures to follow for replacing timber decking with cast-in-place (CIP) concrete decking include removing damaged material, rebuilding the substructure and deck equipment, constructing the formwork, placing the concrete and removing formwork.

Removal of Damaged Material

3. In the case where portions of the timber deck are repairable, it is important to remove all damaged lumber as well as wood that is suspected of being damaged and to keep only good quality materials in the structure to be repaired. Not only should the damaged materials be removed, but the old structure should be removed to the boundaries of the substructure beam or pile cap in order that new concrete can be properly supported by these members. All timber members that are removed should be cut flush and clean to provide good quality construction joints for the new concrete. It is most likely that portions of the substructure are also damaged in areas where the deck has been damaged, and these members should also be removed and replaced.

Rebuilding the Substructure and Deck Equipment

4. The substructure of a timber pier/wharf varies depending on the type of construction. The substructure of solid type deck construction probably consists of fill material and should be replaced with similar material. Openly constructed piers/wharves will probably be supported by timber piles

and pile caps followed by stringers and decking. Damaged piles can be repaired by one of several methods mentioned in Technical Report GL-86-21 (Eastport International Inc. 1986).* Piles removed or no longer used can be replaced by other timber piles or by precast concrete piles. Damaged pile caps could be replaced by CIP concrete. The methods for forming these structural units are identical to deck CIP concrete and are discussed herein.

5. The hardware necessary for outfitting a pier/wharf consists of chocks, cleats, bollards, mooring posts, and other hardware used to moor a ship to the dock. These attachments must be properly anchored to the new deck and to the substructure in order to perform their designed task. It is important that the anticipated horizontal and vertical loads which these hardware will experience are properly transferred to the foundation. If the entire deck is being replaced with CIP concrete, then the proper amount of reinforcing steel should be placed around the location of the hardware insert and tied into the stringer and pile cap before placing the concrete. If the replaced substructure is wood, then additional stringers should be placed beneath the hardware placement area in order to transfer loads to the pile caps. If a portion of the timber deck remains and a CIP deck is placed over it, the deck hardware should be installed at the proper elevation to adjust for the thickness of the concrete above the old deck elevation. Army Technical Manual TM 5-360 (Headquarters, Department of the Army 1964) provides information for typical types of pier/wharf hardware.

6. The concrete that is placed on the existing piles and substructure is a heavier material than the structure it is replacing. This is a concern because the piles and substructure may not be designed to carry such an increase in load. The substructure must be structurally strong to support additional loads without bending, buckling, or crushing its members. The unit weight of concrete is approximately four times greater than wood products. The design of the substructure and the foundation should be checked to determine if they can withstand the additional load, and if found to be inadequate, substructures should be redesigned for additional loadings.

7. Piles can be strengthened by reducing their unbraced length to minimize the buckling associated with additional loads. This is accomplished by

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* References cited in the Appendixes can be found in the references at the end of main text.

providing additional bracing to the pile either through cross bracing to another pile or by bracing to the foundation by means of a batter pile, or by bracing from the pile to the deck. The best location to brace the pile is the intermediate point of its longest unbraced length. This may be underwater and inconvenient; however, it is the point that provides the pile the greatest capacity for additional load. If the bracing point is not convenient, the nearest accessible location should be used. The bracing should be added in the plane that withstands the greatest bending moment. If possible, bracing should be added to prevent buckling against both the major and minor bending axis of the pile, but if limited time and resources prevent this, bracing should be placed against the major axis. Buckling calculations should be determined and engineering decisions made whether additional piles should be driven in addition to the bracing.

8. Additional piles can also be driven to increase the load-carrying capacity of a structure. These piles can be driven adjacent to existing piles, or at an intermediate location between piles in a given bent to support added loads. If possible, the newly driven piles should be precast concrete piles, since they are stronger and last longer than timber piles.

9. Timber pile caps can be replaced by CIP concrete caps and have greater strength and last longer than their timber equivalent. Their design depends upon the constraints of the individual project; however, their construction essentially consists of constructing formwork around the in-place pile groups, providing the proper reinforcing in the cap formwork, and placing the concrete into the forms. Army Technical Manual TM 5-302 (Headquarters, Department of the Army 1986b) provides several standard designs.

10. Pile caps, stringers, and decking can be cast integrally using CIP concrete since the concrete conforms to the shape of any formwork into which it is placed. Casting deck and substructure at the same time not only saves time and formwork but also provides a stronger structure since no joints are between members.

Formwork for Subdeck and Deck

11. The intent of formwork is to contain and support the fresh concrete until it has hardened to the point where it no longer needs containment, and it has sufficient strength to support its own weight and the weight of any

materials or other structures placed on it. The forms for concrete structures must be tight, rigid, and strong. If forms are not tight, they will lose mortar which may result in pockets where there is no cement and only aggregate, or a loss of water that causes sand streaking. Since the formwork probably needs to be made from onsite materials or those obtained from within the host country, the problem of good formwork is compounded.

12. In general, the formwork for all port structures should be designed, constructed, and maintained in accordance with ACI 347-78 (American Concrete Institute 1984e). The following paragraphs are intended to point out some of the areas where the use of existing materials or available techniques may cause less than desirable concrete results.

13. Although there may be some construction material available for theater construction, it is not known whether there will be adequate lumber available to use as formwork. In this situation, the engineer may have to improvise and use available wood materials for the construction of forms that will be necessary to contain repair concrete. The problem with concrete construction using scavenged materials is that the available materials are not always of the best quality.

Sources of materials

14. The first source of lumber for formwork should be from military supply. It stocks the best quality lumber for the job, which has been obtained from suppliers in the USA, and a certain amount of quality assurance can be assumed. The supplies obtained from a host nation are the next echelon of supply to use. Supplies from some countries may not meet the standards which are required for good quality concrete formwork, and the materials may cause problems with the final concrete product. However, the use of lumber processed for formwork is better than constructing formwork from left-over materials. The final source of materials is wood or metal structural members that can be scavenged from structures on site or in the local area that are not of any use to the mission of the port. These materials will be the least desirable since they were not designed for the job. They were not properly seasoned, and the proper hardware to make the formwork perform correctly may not be available.

15. An alternative to using scavenged materials is to utilize the existing timber deck structure to perform as formwork for the placement of a CIP concrete deck. Since the deck will not be properly constructed to serve

as formwork, it should be prepared to adequately contain the concrete placed on it. This consists of caulking all cracks and spaces in the decking to make it mortar tight, and strengthening the substructure to accept the weight of the concrete.

Lumber

16. The lumber used should be straight, have flat edges, and be structurally sound to construct good quality formwork. With lumber purchased from a supplier and intended to be used for formwork is not always a problem; however, when one must improvise and use materials found on site, problems can arise. When using scavenged materials, their qualities should be checked and assured. Old boards taken from timber warehouses or structures should be checked to ensure the boards are straight. If the boards are not straight, they should be straightened with a table saw. It is recommended that boards used in formwork be partially seasoned to prevent them from changing shape during the construction work. Wood that is too green will warp when it has a chance to dry, and wood that has been too seasoned will swell due to water added from wet concrete. If it is not possible to use the properly seasoned woods, existing supplies must be used and the best possible precautionary measures taken to ensure good formwork.

17. With overly seasoned woods, such as those probably located in abandoned buildings, care must be taken to prevent swelling. The old wood absorbs moisture from the concrete if not prevented. This causes the wood to change shape and possibly causes gaps in the formwork. Not only does it change the shape of the wood, the water taken from the concrete mixture changes the water-cement ratio of the concrete adjacent to the form and possibly causes the concrete to weaken in that region. These woods can be presoaked with water to alleviate swelling changes prior to cutting the forms. This minimizes the shape changes and provides the extra water that would be absorbed from the concrete-mix water. Unless it is absolutely necessary, salt water should not be used since it contributes to the salts present that can cause reinforcement corrosion.

Stresses

18. Formwork has to support very heavy loads before the concrete has matured enough to support its own loads. It is therefore important that form woods be oriented to support the maximum load possible. The load supporting members of the formwork are the shoring, wales, bracing, and form facings.

Form facings

19. Normally, the facing of forms is constructed of plywood. The most common varieties are 3- and 5-ply panels. The greater the number of plys, the greater the load supporting capacity. Figure B1 shows the correct way to orient plywood. The face grain of the plywood should always run perpendicular to the supports. This orientation of plywood placed in the strong direction ensures that the form facing has a greater load supporting capacity.

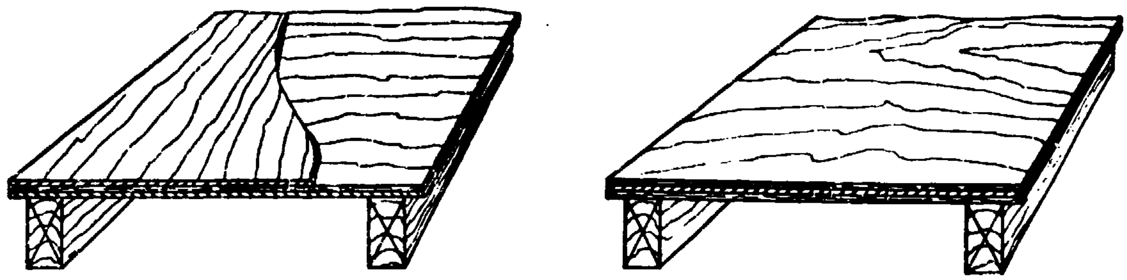

Figure B1. Plywoods used the correct way with face grain perpendicular to the span and the wrong way with face grain parallel to the span (Courtesy of American Concrete Institute)

Wales

20. Wales are bending members that are used to keep the form facings from bending outward under the great stress of wet concrete. They are nailed to the outside of the plywood forms, generally in horizontal rows, and frequently spaced to prevent the plywood from bending. Wales are commonly cut from 2 by 4's or larger dimensioned cut lumber. They should always be oriented such that the smallest dimension face of the wale is in contact with the form facing (Figure B1). This orientation ensures the wale to support the greatest bending loads for its cross section. Sizing of wales to prevent bending of the form facing is a function of the loading that wet concrete exerts on the plywood and must be taken into consideration. The engineering staff assigned to the work party should design the wales for all construction situations.

Bracing and shoring

21. These members are intended to transfer the loads of the formwork and the concrete to the foundation. They perform their task by supporting axial loads along the bracing or shoring member. Bracing and shoring are usually lumber of the 2 by 4 or 4 by 4 variety. Axially loading these members subjects them to bending forces which can cause the bracing or shoring to

buckle under eccentric loads. The engineer can usually determine where the buckling may occur and add structural members to prevent it. All bracing and shoring should be properly placed to reduce the unsupported length and provide additional stiffness to the load supporting capacity. All brace or shoring member splices should be braced at the splice in both bending directions to reinforce the splice at the weak point.

Form fasteners

22. All wood formwork must be constructed with form nails or similar fastening devices that will hold the formwork in place until the concrete has hardened. Form nails should be in good supply from Army stocks. Bolts and nuts to fasten wales at the corners of formwork should also be in good supply. Column formwork can be banded together in the absence of the proper wale fasteners. The banding material can be anything from rope to chain or anything that will hold the form facing in its desired location. Materials that stretch such as ropes should be tightened to take up all the stretch before the concrete is placed. Ropes can be soaked with water before being placed and as it shrinks when drying, the band becomes tighter. Form ties may be in less supply than other types of form fasteners. Rods and nuts fitted with large washers can be substituted for these fasteners to prevent the formwork from bowing due to the pressure of the wet concrete.

Types of Construction

Monolithic beam/slab construction

23. If the entire deck area has been damaged and needs to be replaced, beam/slab construction is the most efficient CIP method to use. The substructure (in this case all elements above the pile cap and, if necessary, including the pile cap) and the deck concrete are all formed and cast at one time thereby making all structural elements monolithic in nature. This type construction has advantages of placing all the concrete at one time and separates the chores of formwork construction and concrete placement. The disadvantage of this method is that formwork construction is more complicated and the placement of the concrete can be difficult when trying to move the concrete to remote corners in the forms.

Concrete wearing surface on timber deck

24. If the timber deck does not need to be removed, then the possibility of adding a strengthening concrete deck to the existing structure is useful. In order to make the best use of the tensile supporting capacities of the wood and the compressive strength of the concrete, the concrete overlay should be anchored to the wooden deck by anchor pins. The anchor pins act to resist any shear movement between the two materials used for the deck. These can be either bolts or spikes driven into the wood and allowed to extend up into the area where the concrete is placed. The wood and concrete composites enhance one another when the concrete has hardened.

Form Construction

25. The objective of constructing forms is to fabricate the forms in a manner that will safely contain the concrete at the proper elevation, in the proper shape, and until it hardens thereby supporting itself. It is difficult to describe proper methods of form construction. Each construction job has aspects that make it different. It is best to construct the formwork in a manner that is familiar to the personnel performing the construction rather than try to train workers to do it another way. The following paragraphs suggest a method of constructing formwork.

26. The following information is presented for forming beam or girder elements, slab elements, and shoring during construction of pier/wharf decks. These elements are the major structural members necessary for replacing a wooden deck with CIP concrete. Column elements, slabs on grade, footings, etc. are beyond the scope of this report and will not be discussed.

Beams and girders

27. Beam formwork consists of sides and bottom of a beam being formed, kicker blocks for resisting bowing of the beam walls, and ledger blocks for supporting any slab formwork as well as bracing and wales. Figures B2 through B4 show some aspects of proper beam form construction. Beams or girders can be intermediate members that span between the pile caps of the old pier, or they can act as the pile cap itself. Intermediate members are formed as shown in the figures. Pile cap members are formed around the piles and use the piles as their shoring members. Beams are formed before any other member in the beam/slab repair area. As shown in Figures B2 through B4, the beam

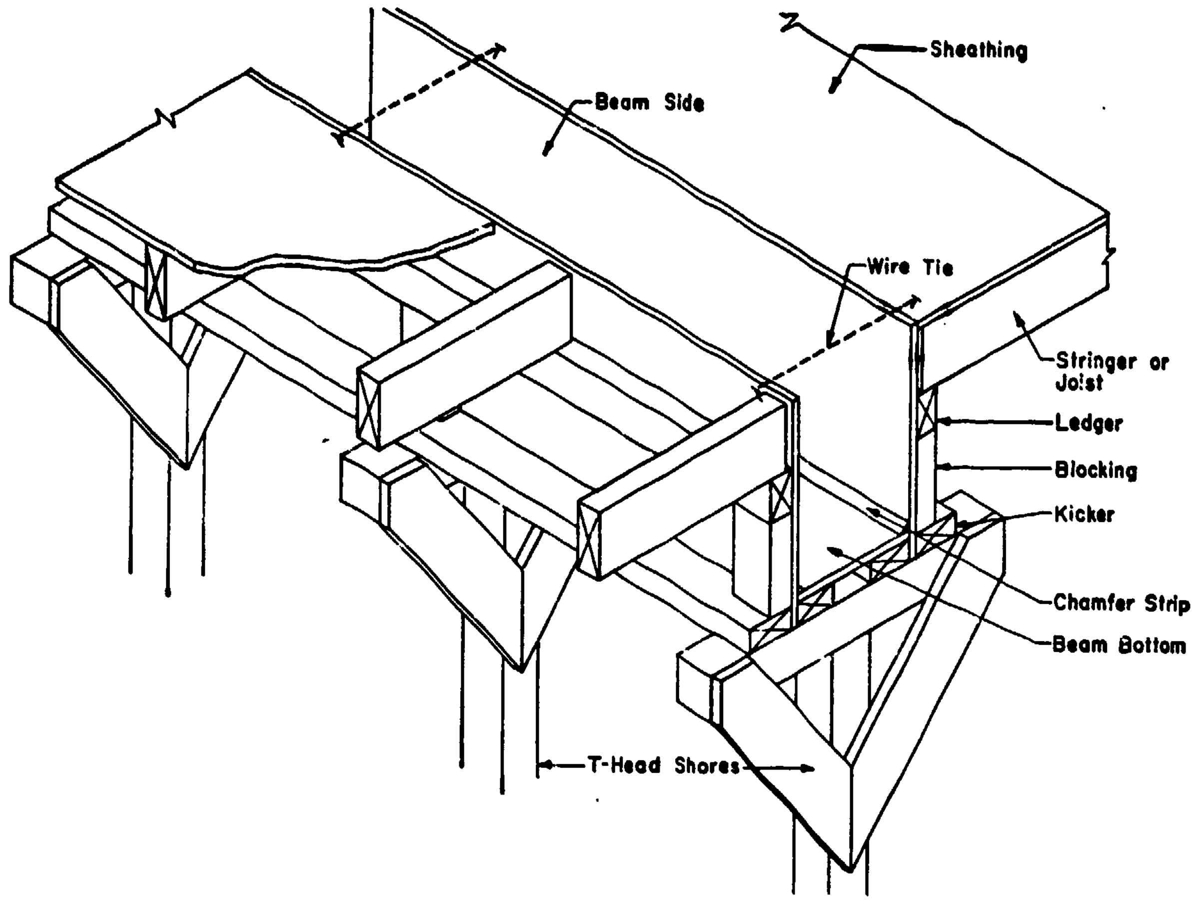

Figure B2. Typical beam form construction showing framing necessary for slab to be placed along with the beam (Courtesy of American Concrete Institute)

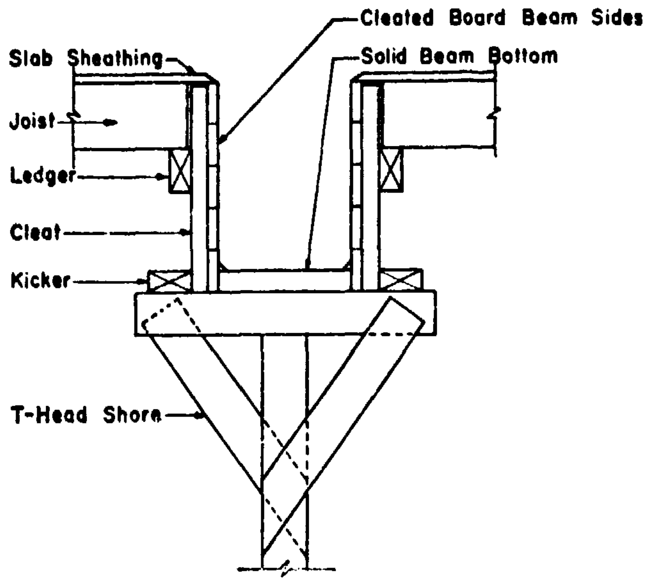

Figure B3. Beam form details where cleated boards form the sides (Courtesy of American Concrete Institute)

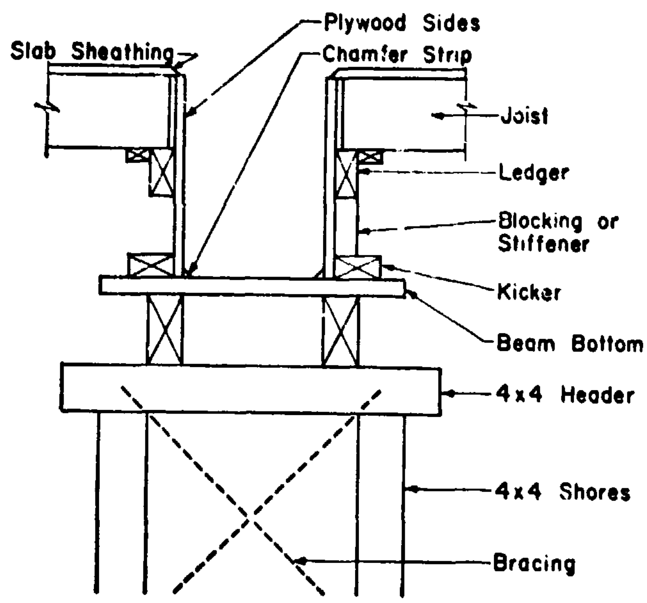

Figure B4. Beam form detail with extended bottom and resting on double 4 by 4 shores (Courtesy of American Concrete Institute)

bottom rests on the shoring header either directly, or carried by stringers placed on the header. The bottom consists of 3/4 in. plywood, but other types of flat lumber can be used if the edges of the lumber fit together. The sides of the forms also rest on the header (Figures B2 and B3) or on the extended bottom (Figure B4). The sides extend up to the underside of the slab that frame into the beam. Stiffeners or blocking (Figures B2 and B4) are nailed to the sides of the beam to strengthen the walls of the beam and add support to the ledgers that are nailed above them. The sides usually consists of plywood, but they can also be constructed from board lumber. If boards are used, cleats (Figure B3) should be nailed to the boards at 2- to 2-1/2-ft intervals. The plywood sides require vertical stiffeners if the beam sides are extremely tall.

28. Kickers and ledgers are longitudinal members of the beam formwork that extend the full length of the beam. Kickers are members (usually 2 by 4's) that are nailed to the side of the beam and to the headers and are used to keep the bottom edge of the side panels from bowing outward under the weight of the concrete. Ledgers are support members that are nailed to the side of the beam to support the joists that carry the loads imparted by the slab formwork that forms into the beams. They are nailed to the sides of the beam at the elevation where the joists can sit on top of the beams such that the top of the joist is at the proper elevation to support the slab sheathing.

29. If the beams being formed are tall, the sides of the beams need to be supported by ties running through the sides of the form. This keeps the walls from bowing outward under the pressures of the fresh concrete.

30. The slab sheathing frames into the top of the beam by resting on the side of the beam formwork. It is usually beveled towards the face of the beam in order to facilitate removal of the formwork and provide a pleasing corner detail.

31. The construction condition at the intersection of the beam and other members into which it forms is important. The length of members used to form the beam should be the clear span of the beam less the thickness of the sheathing of the members into which it will frame and less about 1-1/2 in. on each end. This allows the formwork to be easily removed without damaging any young concrete. The gap that is left between the beam and the other member can be filled with a beveled piece of 2 by 4 or a metal angle strip.

Types of slab construction

32. There are a number of different types of slab construction. The most common are beam/slab construction, flat slab construction, and pan slab construction. If full sections of a pier are being replaced, either pan slab or beam/slab construction is the most expedient and the strongest. If the pier is being surfaced or strengthened, the flat slab methods can be used.

33. Beam/slab construction. Slab sheathing to support CIP concrete rests on beam joists which rest on ledgers nailed to the side boards of the beams which are also to be cast as part of the beam/slab construction. The slab sheathing which frames into the beam is shown in Figure B5. For the smaller spans in beam/slab construction, the slab is supported on just the joists. For longer slab spans or where the slab is particularly thick, intermediate supports are needed to prevent the formwork from sagging. Figure B5 shows intermediate stringers supported on shores running perpendicular to the direction of the joists. The top of the stringers should be at the same elevation as the top of the ledger such that the joists receive the same amount of support wherever there is a stringer. If stringers are needed for support, the shoring is placed first, the stringers are placed on top of the shoring, and the joists on top of the stringers. The sheathing is then laid on top of all substructure. The joists are not always nailed to the supports, particularly if it is desired to make stripping of the forms an easy chore. It is important to make sure that the joists and any stringers do not turn over or rotate from their intended locations in the formwork. This is potentially dangerous if the joists are tall relative to their thickness.

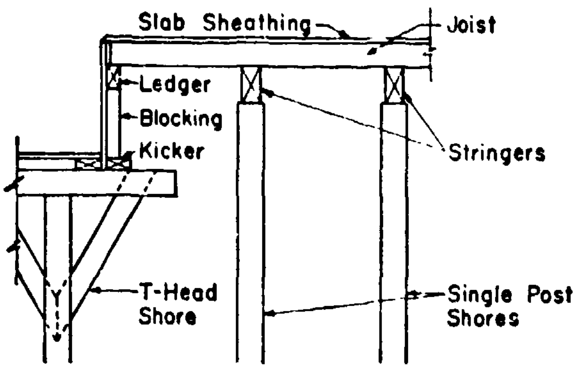

Figure B5. Slab sheathing supported on stringers and shores showing detail of connection to beam formwork (Courtesy of American Concrete Institute)

34. The slab sheathing can be any material that supports the weight of the concrete. In most instances plywood sheathing will be used, but it can be flat boards or other types of flat lumber.

35. In the case of repair to pier/wharf decks, the repaired area will probably extend from one major substructure support to the next one that is in useable condition. If this creates a condition where new concrete is being placed on either side of a new beam, the loadings on the beam formwork should be similar on both sides of the beam. However, at the periphery of the repair area, the slab formwork will frame into a beam that does not have a slab framing into the other side. For this condition, the beam should be designed to account for this one-sided weight such that no beam rotation takes place.

36. Pan slab construction. Pan slab construction consists of a series of regularly spaced concrete joists supporting a thin slab, all of which are cast in place monolithically. It is similar to beam/slab construction in that there are beams (referred to here as joists) and a slab; however, the slab is thinner in pan slab construction, and there are more joists to compensate for the thinner slab. The versatility of this type of construction lies in the two types of pan slabs. When the joists are constructed such that they all run parallel to each other, it is called one-way pan construction. This is ideal when the clear span between supports is large in one direction and short in the other. When the clear span between supports is large in both directions, two-way pan construction with joists running in perpendicular directions in a waffle pattern provides the needed support. Figures B6 and B7 show one- and two-way pan construction formwork. In Figure B6 the construction is for one-way pans. The beam form rests on shoring, and timber soffit planks span between adjacent beam forms. These soffit planks are spaced so that the edges of the pans can be nailed to the planks and the planks act as the bottom member of the joist form. The pans are placed on the soffit planks and act as forms to support the slab above and form the sides of the joists. One-way slabs are constructed from pans oriented perpendicular to the supporting beams. As in Figure B6, the soffit planks are supported by stringers and shores at intermediate locations between the beam supports in order to prevent sagging of the slab forms.

37. The pans can be fabricated from board lumber or can be specially fabricated fiberglass or steel pans. Those shown in Figure B6 are constructed as segments and fitted together; however, whole pans can be constructed and

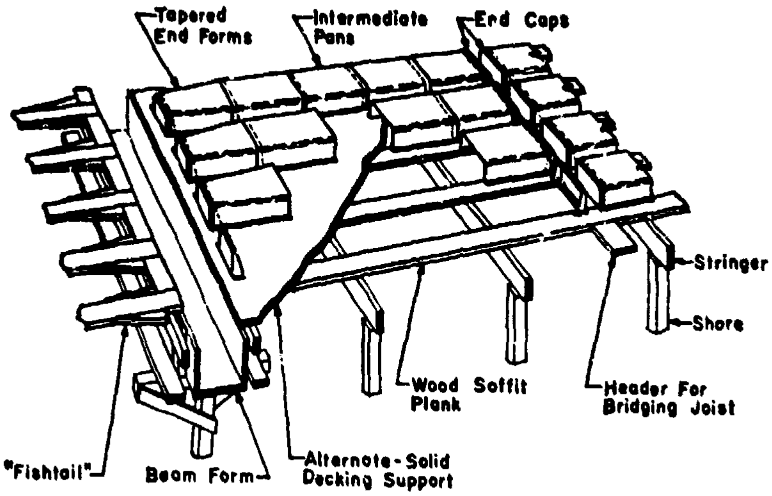

Figure B6. Example of form construction for one-way pan slab showing soffit plant and solid deck support (Courtesy of American Concrete Institute)

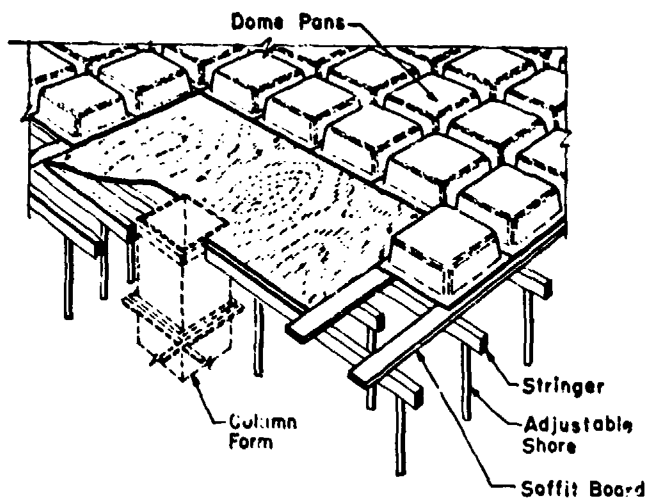

Figure B7. Example of two-way pan slab formwork (Courtesy of American Concrete Institute)

placed on the soffit planks. The prefabricated pans are easier to remove after the concrete has hardened, but they are also more expensive.

38. An alternative method of supporting the pans is to replace the soffit planks with solid deck sheathing as shown in the left side of Figure B6. This is more expensive, but it provides a more convenient working surface for personnel when nailing down the pans.

39. Where segmental pans are used, construction first starts with chalk marking of the soffit plank to ensure alignment of the pans. The end pans are first nailed to the soffit and construction works towards the middle of the span, overlapping each segment pan by 2 to 3 in. If necessary, a filler pan is added in the center area to complete the row. This is continued until the entire span has been covered.

40. Figure B7 shows the two-way pan slab formwork. Since the joists are formed in both major directions, the slab has better bending resistance in both directions and the thickness of the slab is thinner than the one-way slab. The forming construction for these pans is similar to one-way pan construction, except the pans are all square-dome pans which are nailed to either soffit boards or solid decking. Figure B7 also shows both soffit planks and solid decking. The solid decking is used to support a thickened slab at the column.

41. In order for pan slab construction to contain enough concrete over the tops of the pans to form a slab, the sides of the repair area must be built up above the top of the pans to a thickness equal to the desired thickness of the slab. Once this has been completed and all inclusions have been placed into the formwork, the slab can be placed.

42. Flat slab construction. Flat slabs are directly supported by pile caps or piles without the aid of any intermediate beams or girders. In repair techniques using cast in place concrete, flat slab construction is probably used only when strengthening an existing deck or when repairing a deck that is only to be used for lighter loading situations.

43. When strengthening an existing deck, the existing deck can be used as the bottom of the formwork to support the fresh concrete. This eliminates much of the construction effort in strengthening the deck. The loading of the fresh concrete should be calculated to determine whether additional strengthening of the substructure, shoring, or bracing is needed. Areas of the deck that have been damaged and need to be replaced must be completely formed to

provide substructure and deck for concrete placement. This can be accomplished by using one of the CIP slab techniques mentioned in prior paragraphs.

44. In situations where new construction of a flat slab is needed, the first chore in formwork construction is to provide shoring and bracing for the area to be placed as a flat slab. Stringers and joists are then placed on top of the shoring and adjusted to the proper elevation for acceptance of the deck sheathing. The deck panels are then placed over the joists. Form siding is placed around the periphery of the repair area up to the elevation of the finished deck.

Shoring

45. The construction of shoring in any CIP formwork situation is critically important to ensure that the forms do not move or sag under the weight of the fresh concrete. Construction over water, as in the case of pier or wharf construction, presents problems which are compounded by limited vertical supports (piles) being spaced wider apart than is customary for shoring members. Additionally, the height of the decking above the ocean floor must be 30 to 40 ft in order to accommodate ships docking at their side. This makes it difficult to transfer load directly to the foundation by means of shoring to the ocean floor. All shoring loads must be eventually transferred to the piles.

46. Beam shoring which support formwork to cast new pile caps can directly transfer load to the piles around which the caps are casted. Figure B8 indicates one method of constructing the formwork to accomplish this goal. In order to accommodate the length of pile that is casted into the cap, holes that are the diameter of the pile are cut in the lumber and the form bottom is placed over the piles. The T-head shore members (the horizontal beam beneath the form bottom and the diagonal bracing lumber) should be nailed into the beam bottom and either bolted or nailed to the pile. This shore detail is repeated at each pile in the pile cap group.

Horizontal shoring

47. Since pile groups are generally 10 to 20 ft apart, they present a very large span where there is nothing capable of supporting any intermediate members such as stringers and shores. In order to keep slab formwork from sagging in these situations, horizontal shoring is appropriate. Horizontal shoring consists of substantial supporting members (such as steel joists, steel channels, or heavy timbers) spanning horizontally between the piles

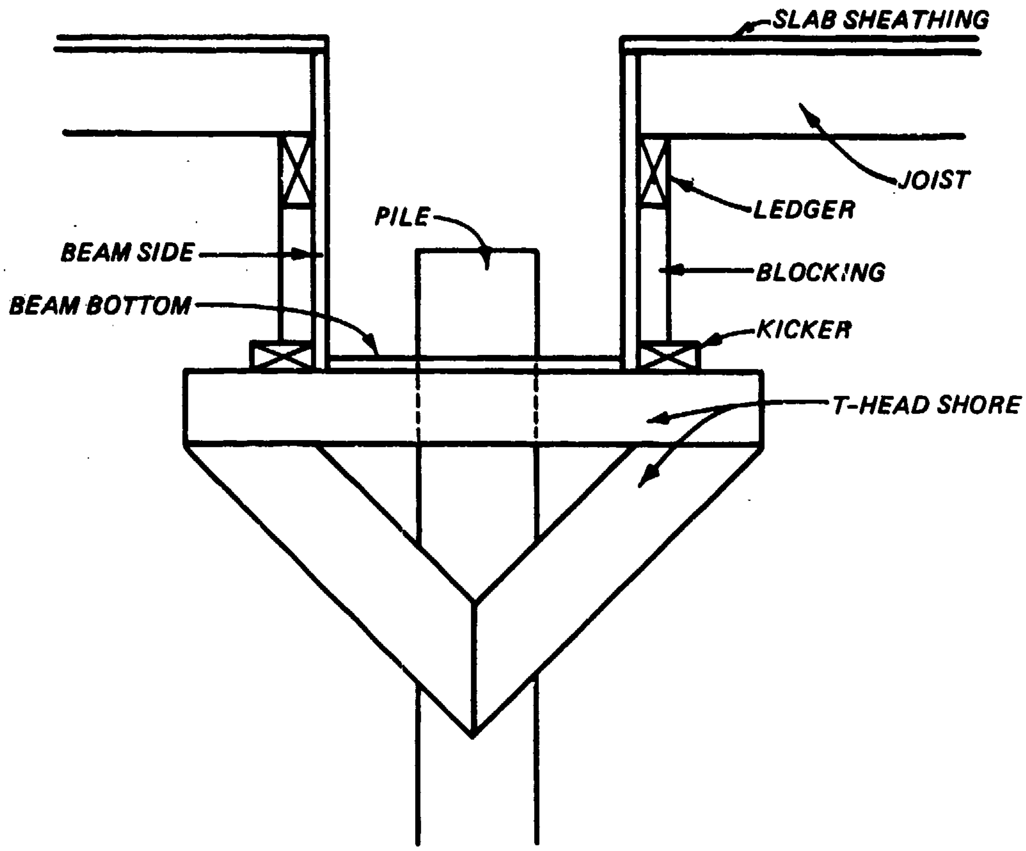

Figure B-8. Detail of shoring for pile cap formwork

below the slab formwork. These members must be bolted to the piles and designed not to sag significantly under the concrete weight imposed by the intermediate shoring that spans between the stringers and the horizontal shoring. Figure B9 shows a sketch of steel channel members acting as horizontal shoring. The channels should be located as high on the pile as possible in order to minimize the length of the intermediate shores. Since relatively large spans will have to be bridged between piling, there is some expected sagging of the horizontal shoring under the weight of the concrete. The intermediate shores should be constructed such that they can raise the slab sheathing to the proper elevation during placement of the concrete. This can be accomplished through the use of shoring that is adjustable or by means of wedges as shown in Figure B10. The wedges should be used in pairs as shown in the figure and should be toenailed to the base plate when they are at the proper elevation. These wedges also aid easier stripping of the formwork.

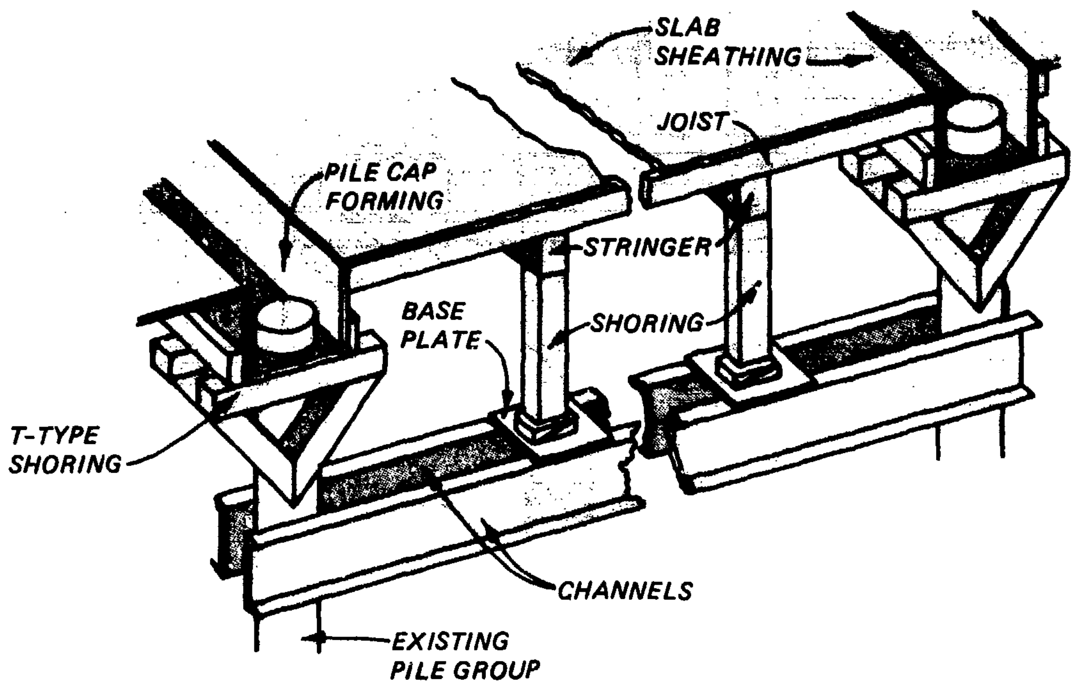

Figure B9. Detail of horizontal shoring using steel channels to support intermediate shoring

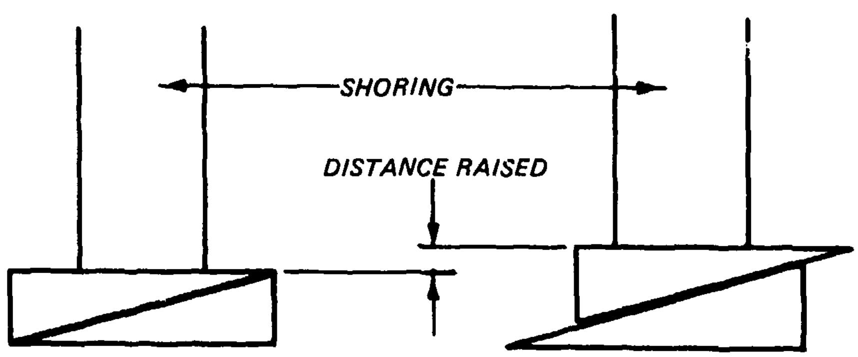

Figure B10. Wedges used to raise vertical shoring

Manpower and equipment

48. The personnel necessary to construct formwork for CIP concrete must have general construction skills. They essentially work with carpentry tools; however, some mechanic tools are required to bolt some of the formwork together. The Port Construction Company should be augmented with a General Construction Platoon from an Engineering Construction Company for the purposes of repairing the deck by means of CIP concrete. The tasks required to construct the formwork and place the concrete are fabrication of the formwork, fabrication of the reinforcing steel cages, formwork preparation, insertion of all inclusions, placement of the concrete, and stripping and curing of the concrete. With this additional help, the formwork for one 20- by 20-ft bay of CIP concrete can be constructed in two working shifts, and construction of an entire 80- by 1,000-ft wharf deck would take an experienced team approximately 4 months. The necessary construction equipment includes carpentry tools, table saws, lifting equipment such as light cranes, and mechanic tool sets for bolting heavy members together. The concreting equipment includes concrete mixing equipment, concrete transportation equipment, vibrators, and finishing equipment.

Preparations for concreting

49. When the formwork has been completed, there are a number of steps in preparing the forms for concrete acceptance. All the joints in the forms should be checked for mortar leakage prevention when the concrete is placed. If mortar is allowed to leak, the areas behind the leaky joints will be honeycombed and lack cement. All elevations should be checked to make sure that all formwork is up to grade, and shores should be adjusted accordingly.

50. Form release agents. The form release agents should be applied to the forms immediately before any reinforcing is placed into the forms. This is required to prevent any of the form release agents from getting on the surface of the reinforcement and reducing its capacity to bond with the concrete. Workers trying to paint the sides of formwork from between reinforcing bars have a tendency to spill paint on the bars. The form release agents can be any material that prevents the concrete from bonding to the forms. They could be oils, waxes, greases, emulsions, or just plain water. They should not be of a nature which would destroy any curing agents that may be placed on the concrete after the forms have been removed, or would retard the bonding of any coatings such as paints or other surface coatings. For example, if the

concrete is painted, oils should not be used because they would leave a film of oil on the finished concrete surface and the paint would not adhere.

51. Inclusions. After the form surfaces have been prepared, all inclusions should be placed into the forms. This includes all reinforcing, ducts, pipes, anchors, bar chairs, deck equipment and any other materials that are to be cast in the concrete. The reinforcement should be located as shown on the engineering plans and the tolerances should be followed as closely as possible.

52. Inspection. Before the concrete is placed, the formwork should be inspected to assure everything has been included: the forms have been oiled, everything is at proper elevation, and the forms are clean and free of dust, dirt, and any other foreign substance that would interfere with the bonding of the concrete. The proper cover over the reinforcement should be checked to make certain that the steel is protected from exposure to the salt water. ACI recommends that cover over the steel be at least 2-1/2 in. to prevent exposure to marine environments.

Placing Cast-In-Place Concrete

53. The requirements for placing repair concrete are generally the same as for the original construction. The new concrete should consist of the same materials (if known) and of approximately the same proportions as the original concrete, unless a specified concrete mixture is chosen.

54. All personnel concerned with concrete work should be aware of the importance of maintaining the unit water content of the mixture as low as possible. The concrete's water content should be consistent with the placing requirement. When indiscriminate amounts of water are added to the mixture, the water-cement ratio is changed, and both strength and durability are adversely affected. Generally speaking, for a given amount of cement, the more water that is combined with it, its strength and durability are lowered. If the original concrete mix is unknown, a new mixture proportion should be determined. The properties of the concrete should be tailored to meet the requirement of the placement. Things to be considered are placability, consistency, strength, durability, density, generation of heat, and time required before the concrete can carry weight. The procedure for selecting mix

proportions is given in ACI 211.1-81 (American Concrete Institute 1984k) and FM 5-742 (Headquarters, Department of the Army 1985a).

Placing operations

55. The basic requirement for the amount of placing equipment and the type of placement techniques, as for all handling equipment and methods, are that the quality of the concrete must be preserved. Sufficient placing capacity as well as mixing and transporting capacity should be provided so enough concrete can be placed in the forms to prevent cold joints from forming. Cold joints are caused by slow delivery of the next batch of concrete. Wherever possible, concrete should be dropped vertically into the forms. Dropping concrete at an angle causes segregation of the coarse aggregates. If necessary, a drop chute made of canvas sheets sewn together will provide a proper path for concrete to be dropped from any significant height.

56. Once the concrete has been placed in the formwork, it should not be moved a great distance horizontally either by pushing equipment or vibrators unless it is certain that all the material is being moved integrally. When placing concrete from a second batch in the vicinity of the first batch, the new concrete should be placed directly against the previously placed concrete. It is better to directly place the concrete in the corners and edges of formwork and work towards the center than to work towards the edges.

57. When spreading new concrete to achieve smoothness, a toothed rake should never be used since this tends to separate the larger aggregate particles from the cement matrix. Only flat tools such as "come alongs" and shovels should be used.

58. For deep concrete placements requiring monolithic construction, each concrete layer should be placed while the underlying layer is still responsive to vibration, and each layer should be sufficiently shallow to permit knitting the two together by proper vibration. Vibration practices are discussed in paragraph 66 of this appendix. The placing of concrete shall be performed in accordance with ACI 304-73 (American Concrete Institute 1984g) unless other military specifications take precedence. If complete compliance cannot be achieved, the field engineer must use his best judgment to achieve properly placed concrete.

Consolidation of concrete

59. A quote from the ACI 304-73 (American Concrete Institute 1984g) is as follows:

> A mass of freshly mixed concrete as deposited in a form or mold is usually honeycombed with entrapped air. If allowed to harden in this condition the concrete will be non-uniform, weak, porous, and poorly bonded to the reinforcement. It will also have a poor appearance. The mixture must be densified if it is to have the properties normally desired and expected of concrete. Consolidation, also called compaction, is the process of removing entrapped air from fresh concrete in the form.

Proper consolidation is not only important from the standpoint of producing strong concrete, but also from the viewpoint of durability of the concrete and providing concrete that will remain strong for a lifetime. The following paragraphs discuss the equipment and procedures necessary for good concrete consolidation.

60. There are several methods of achieving consolidation of CIP concrete. The most simple of these is to rely on gravity to consolidate the aggregate and cement into a dense mass. This is only useful if the concrete is fluid enough, when placed, that it can overcome the internal friction forces to seek its own consolidation. Through the use of chemicals known as water-reducing admixtures (superplasticizers), concretes can be produced that have self consolidating properties and will consolidate under the forces of gravity. Most concretes are so stiff that they need assistance in becoming consolidated. Consolidation by vibration is used to assist the forces of gravity in compacting the concrete. Vibration of the concrete causes the otherwise stiff mass to liquify and overcome the resistance of internal friction such that the aggregate particles and the cement matrix can come together in a dense mass. Vibration also allows the entrapped air voids to work their way to the surface of the concrete and escape the mass.

61. There are several types of mechanical vibrators that are commonly used. Among them are power tampers, centrifugation, shock or vibration tables, and internal and form vibrators. Under theater of operation conditions and supply, the only types that are expected are internal and form vibrators. In the vast majority of field work, internal vibrators are used to consolidate CIP piles or columns and slabs or decks that contain reinforcement and are thicker than 6 in. Vibrating screeds consolidate all thin slabs. External form vibrators can be used on walls and columns in areas where internal vibrators are ineffective.

62. The most common of the internal vibrators is the flexible-shaft type poker vibrator. A schematic is shown in Figure B11. This vibrator is driven by either an electric or gasoline motor and causes a long flexible shaft to rotate an eccentric mass in the head of the vibrator. This produces a frequency that liquidates the concrete and provides the required consolidation. Other types of internal vibrators are electric-motor-in-head type and

Figure B11. Flexible shaft vibrators (Top: electric motor driven; middle: gasoline engine driven; bottom: cross section through head) (Courtesy of American Concrete Institute)

air pressure driven vibrators, but these are not as common as the flexible-shaft variety although their principle of operation is much the same.

63. Form vibrators are also commonly used. They provide vibration in areas where the internal vibrators cannot reach. These are attached to the outside of the formwork thereby vibrating the formwork rather than the concrete. The vibrations are passed through the formwork and into the concrete inside the form. Typical types are rotary, reciprocal, electromagnetic, and pneumatic. The most common is the rotary type.

64. Surface vibrators are used to efficiently consolidate large flat expanses of concrete such as slabs or pavements. They perform vibration from the top surface rather than from insertion, or from the outside of formwork. The vibrating screed shown in Figure B12 is most commonly used. It consists of a long single or double beam that is supported on skids or rollers that glide along the edge of the formwork containing the concrete. The vibrator is attached on top of the beam. The vibrations are passed along the beam to the concrete, and due to the shallow depth of the concrete, the vibrations are distributed throughout the concrete. This type of vibrator should only be used on very shallow depth placements on the order of 6 to 8 in. deep and generally unreinforced. For deeper, slab-like placements the consolidation should be accomplished by internal vibration.

Figure B12. Single beam screed powered by gasoline engine (Courtesy of American Concrete Institute)

65. Form leakage. In conjunction with consolidation of the concrete, the forms must be tight fitting. The temporary liquidation of the mortar due to the vibration causes forms to leak at the seams thus causing sand streaking of the concrete and general weakness of the concrete near the seam. For detailed formwork information, see paragraphs 25 through 52 of this appendix.

66. <u>Vibrator practices</u>. It is recommended that concrete placement depths be limited to 12 to 18 in. when internal vibration is used for consolidation. Concrete should not be moved laterally with the vibrator. The vibrator should be inserted into the fresh concrete vertically and allowed to fall through the concrete under its own weight until it is approximately 6 in. below the level of the current layer. It should be held at this location for between 5 and 15 sec, depending on the consistency of the concrete mixture, and then withdrawn vertically. This process should be repeated by moving the vibrator such that the areas of the concrete that are affected by the vibration overlap and until the entire layer has been completely consolidated.

67. Proper practices using form vibrators are difficult to achieve and depend on such things as form shape, form thickness, thickness of the concrete layer, workability of the mixture, size of the vibrator, and the time of vibration. The vibrators must be placed on the side of the form at a spacing such that all the concrete in the forms is vibrated. A recommended starting place is to place the vibrators 4 to 8 ft apart and inspect the forms by running the hand across the surface to feel the vibrations. This will indicate the location of greatest vibration. Form vibration requires longer lengths of time to ensure proper consolidation. Proper vibration requires from 2 to 30 min, depending on the thickness of the concrete layer. Adjustments in the practice can be made after inadequate vibration determinations have been made.

68. In very heavily crowded formwork, such as one congested with many reinforcing bars, concrete may be impossible to properly consolidate with internal vibrators because the vibrator may not traverse between all the bars. The form vibrator can be used to vibrate the reinforcement bars and transfer the vibratio to the concrete through the bars. This procedure is performed by attaching the form vibrator to one of the bars. The vibrator needs to be moved to other reinforcement bars to ensure full consolidation. The time span for each vibrator placement varies depending upon the density of the bars and the stiffness of the concrete. The recommendations outlined here are further discussed in ACI 309-72 (American Concrete Institute 1984i).

Concrete Curing

69. Protection of the freshly placed concrete should begin immediately after it has been placed in the forms. The first protective action is proper

curing techniques. Curing is the process of retaining the moisture in the fresh concrete so that it can complete the hydration of the cement, and protecting the concrete from adverse temperature conditions. If the moisture in concrete is allowed to evaporate, there is less free moisture in the mixture to hydrate the cement, and consequently less cement hardens. This results in concrete with reduced designed strength. If the temperature is not above a favorable level, then the possibility of damage to the young concrete from freezing can cause it to be weak and lack durability.

70. The environment associated with a port is not the most ideal for producing high quality concrete. First, there are probable higher than normal breezes usually laden with salt saturated moisture. This transfers salts into the concrete and initiates corrosion of the reinforcement. In winter months the temperature of the sea water usually does not go below freezing; the ambient temperature may be below that which is recommended for good concreting. To counteract these conditions, good curing practices must be observed.

71. There are two types of curing techniques that help retain the moisture in the concrete: frequent application of water to the surface of the exposed concrete and sealing the surface with a nonpermeable membrane to prevent the escape of the existing water. Both techniques are discussed in greater detail in ACI 308-81 (American Concrete Institute 1984j).

Water curing

72. This technique is very water-use intensive. It requires the continuous application of water to the surface of the concrete. Surface water application helps to prevent the evaporation of the water in the concrete. It is assumed that a port would contain an abundance of usable water from the sea. This unlimited amount of sea water contains salt which should not be applied to concrete containing steel reinforcement. Potable water near the port, water from nearby fresh water lakes (which would have to be trucked or pumped to the site), or desalinated water produced by the Army are possible water sources for concrete curing. With the possible exception of the potable water supply, the other fresh water supplies are expensive and consumptive of fresh water that could be used for other purposes. Unless usable water is available in abundant quantities, it is recommended that sealing methods or one of the following water curing methods which do not use great quantities of water be used.

73. Spraying. Water continuously sprayed on the surface of freshly placed concrete prevents the moisture evaporation in the concrete. The method consists of spraying a fine mist of water on the surface of the concrete as soon as the surface of the concrete has lost the sheen of water associated with the finishing of the concrete surface. The spray is produced from a hose set to fine spray or from a soaker type hose which is perforated with many tiny holes for soaking lawns. This method uses large quantities of water.

74. Ponding. If the concrete to be cured is a flat slab or deck, dams can be constructed around the perimeter and the slab flooded with water. This also uses large quantities of water, and has the added disadvantage of placing a large uniform loading on the concrete at its weakest strength condition. There is also the possibility that the dam may not hold the water for the desired time frame, and the protective layer of water could disappear.

75. Coverings of saturated materials. There are numerous materials that are cheap, available, and absorptive of water which can be placed over the fresh concrete and soaked to prevent the evaporation of mix water. Fabrics such as burlap, cotton cloth, and rugs can be used to keep the surface of the concrete wet. These materials must be clean of any substance that can harm the concrete such as sugar which retards the set of cement or fertilizers which can corrode the reinforcement. Heavier materials could retain the water, and fewer applications of water would be necessary. If the materials are thin, double layers would retain the moisture more efficiently. Strips are used to hold the layers in place during high winds. Layer edges should be overlapped to improve moisture retention.

76. Wet sand or moist earth can also be spread on the surface of the concrete to be protected; however, these materials can only be used on flat surfaces, and when filled with water they also add a significant weight to the structure. Large amounts of organic matter in soil can also cause retardation of the hydration of the cement and should be avoided.

77. Sawdust is a material that is normally available in construction areas and can be spread on flat surfaces and soaked with water for curing. The greatest caution here is to be sure that there is not an abundance of tannin in the wood. Tannin is a slightly acidic material found in some woods which is injurious to concrete hydration.

78. Most of the materials that retain water will afford the cheapest method of providing a curing medium, since the longer a material retains moisture, the fewer times water would need to be added.

Sealing methods

79. The water in a concrete mix can be conserved by providing a barrier that prevents its escape. There are two types of sealing methods that are available; plastic films, and membrane forming compounds.

80. Plastic films. Plastic films consist of any sheet of plastic that is free from holes which can be placed over the fresh concrete to retain its moisture. The minimum thickness of the plastic sheet should be 0.004 in. such that it resists tearing or puncturing under minimum force. This method of curing is expensive due to cost of plastics. It is also not likely that plastic material would be available in an Army theater and is not readily available through supply. However, if it can be acquired, it can be reused a number of times without damaging its moisture barrier properties. As soon as the concrete can support the plastic without being damaged, the plastic should be placed directly on the finished surface and weighted with bricks or other heavy objects. The edges of individual pieces of plastic should be overlapped and weighted to prevent moisture from escaping at these locations and to prevent movement by the wind. In cold climates the use of black plastic is preferred for its heat absorption properties, and in warm climates where it is not necessary to retain the heat, clear or white plastic sheet is preferable. Plastic can be wrapped around vertical members after the formwork has been removed; however, due to its rigidity it has the disadvantage of not being usable on the underside of concrete slabs such as a concrete pier/wharf. In this situation the use of membrane forming compounds is more advisable.

81. Liquid membranes. Membrane forming compounds are chemical mixtures which are sprayed on the surface of freshly finished concrete. Due to their properties, compounds form a continuous membrane that prevents water vapor loss. They are quick to apply and very useful on all types of concrete surfaces. After one application, they last until the concrete no longer needs curing protection.

82. Membrane forming compounds are applied either by hand or with a power sprayer using between 75 and 100 psi of pressure. They should be applied as soon as the surface water has disappeared from the finished surface and before the spray can be absorbed into the concrete. They should be

applied according to the manufacturers' directions which are generally printed on the side of the delivered container. They can be used on the vertical sides of concrete members after the formwork has been removed to continue the curing process until it is no longer needed. Because these compounds form a barrier on the surface of the concrete, they cannot be used to protect any concrete that will have additional concrete placed on top of it or any surface that will obtain a coat of paint or an application of other surfacings. The membrane prevents any bond between the concrete and the surface application. If a moisture retaining curing compound is applied, the material should meet the required specification presented in ASTM C 309-81 (American Society for Testing and Materials 1981).

Minimum curing requirements

83. It is important to maintain curing for as long as possible. If there is no hurry to repair a structure, the curing should be left in place for as long as 28 days. However, this is not the case in construction practice because forms must be removed and the work continued. As a minimum, the curing medium should be left in place for at least 7 days when using Type I cement, 14 days when using Type II cement, and for at least 3 days if Type III cement is used. Type III is a rapid hydrating cement, and Type II is a relatively slow hydrating cement compared to Type I which is the most common and most likely to be stocked in Army supplies.

Temperature considerations

84. For concrete piers/wharves, either method of curing (water spray or membrane) is applicable while the temperature is within an acceptable range. While 7 days is the minimum length of time for curing if Type I cement is used, these times should be adjusted if the temperature is at any extreme. In cold weather concrete normally hydrates at a slower rate, and 7 days will not be sufficient for proper curing. Concrete also needs to be protected against freezing. Conversely, in hot climates the concrete hydrates at a much faster rate, and curing occurs in a shorter period of time. Recommendations for curing in various temperatures are discussed in ACI 305R-77 and ACI 306R-77 (American Concrete Institute 1984d, 1984b) and should be followed.

Protection against temperature extremes

85. In cold weather the concrete must be protected against freezing. At temperatures below 50° F the strength development is moderately retarded, below 40° F it is heavily slowed, and below 32° F it almost stops. In

addition, the water in the mixture will freeze and expand which puts very high tensile stresses on the concrete. At this stage the concrete has very little tensile strength. Fortunately, the hydration process gives off heat as a by-product. This heat can be conserved in cold weather to help keep the mixture temperature above 50° F. Styrofoam panels can be cut to match the outside of the formwork and attached before placement, or polyurethane foam can be sprayed on the outside of the form. Both act as insulators for the concrete. Vinyl blankets filled with insulating materials can be wrapped around columns and smaller beams formwork. Mineral wool or cellulose can be used to stuff these blankets. Straw can be piled on flat surfaces to act as an insulator.

86. Heated enclosures can also be used to keep the temperature above freezing. Live steam, forced hot air, or electric radiant heating can be used in these enclosures to provide the necessary warmth. If the heat is provided by combustion, both carbon monoxide and dioxide gases are generated. The former is toxic to human life, and both are detrimental to concrete through the formation of calcium carbonate in the surface regions of the concrete. Special care should be taken to vent these gases to the open air.

87. In hot weather efforts should be made to keep the concrete cooled. The effects of high temperatures and winds on both fresh and hardened concrete include increased water demand, loss of slump, decreased workability, plastic shrinkage cracking, decreased durability, and numerous other side effects described in ACI 306R-77 (American Concrete Institute 1984b). The concrete can be cooled by using cooled mixing water (or ice as a partial substitute for the water) or the placement area can be kept in the shade by erecting tents and wind barriers to keep the wind from causing increased evaporation.

Quality control

88. The purpose of quality control of concrete placement is to assure compliance with strength specifications and to measure the variability of concrete strength over the course of the placement schedule. Concrete, being a hardened mass of several materials, is subject to strength variations due to the influence of numerous variables associated with these materials. Characteristics of each of the ingredients may cause variations in strength of the resulting concrete. Variations may also be introduced by practices used in proportioning, mixing, transporting, placing, and curing. In addition to the variations which exist in the concrete itself, test strength variations are

also introduced by the fabrication, testing, and treatment of test specimens. These variations must be accepted, but concrete of adequate quality can be produced with confidence if proper control is maintained, test results are properly interpreted, and their limitations are considered. This is achieved through quality control.

89. Since concrete is a material whose strength and other properties are not precisely predictable, test cylinders from a given mixture indicate considerable variability in the reported strength. In reinforced concrete design the concrete members are proportioned and the amount of reinforcement is chosen using a specified compressive strength, f'_c, (28-day compressive strength). The strength of a concrete mixture is obtained from compressive tests on standard cylinders (6- by 12-in. cylinders) taken from samples of the field placed concrete. Because of this variability of strength from cylinder to cylinder, concrete mixtures must be designed to provide an average compressive strength, f_{cr}, greater than the design value f'_c in order to assure meeting this required strength a satisfactory percentage of the time.

90. The ASTM standard, ASTM C 94-83 (American Society for Testing and Materials 1983b), recommends that a testing procedure be used that will not allow more than 10 percent of the strength tests to have values below design strength, f'_c. The American Concrete Institute (ACI 318-83) (American Concrete Institute 1984a) recommends two different criteria. Once the test data become available, they recommend that the frequency of occurrence of the average of three consecutive tests fall below f'_c less than once in 100 tests. Their other criteria requires that the probability of any random sample test being more than 500 psi below f'_c be less than 1 in 100.

91. The mixtures should be proportioned for average strengths, f_{cr}, according to the following formulas:

$$f_{cr} = \frac{f'_c}{1-tV} \tag{B1}$$

$$f_{cr} = f'_c + tS \tag{B2}$$

where

f'_c = specified design strength

f_{cr} = required average strength

t = a constant depending upon the proportion of tests that may fall below f'_c

V = forecast value of the coefficient of variation expressed as a fraction

S = forecast value of the standard deviation

The constant t can be taken from Table B1.

Table B1

Values of t

Percentages of tests falling within the limits $\bar{X} \pm t\sigma$	Chances of falling below lower limit	t
40.00	3 in 10	0.52
50.00	2.5 in 10	0.67
60.00	2 in 10	0.84
68.27	1 in 6.3	1.00
70.00	1.5 in 10	1.04
80.00	1 in 10	1.28
90.00	1 in 20	1.65
95.00	1 in 40	1.96
95.45	1 in 44	2.00
98.00	1 in 100	2.33
99.00	1 in 200	2.58
99.73	1 in 741	3.00

92. As a specific example, assume ACI criteria of one random test in 100 being greater than 500 psi below design strength. Assume design strength is intended to be 5,000 psi, and a coefficient of variation of 15 percent is designated. Using the coefficient of variation method (Equation B1), and selecting t from Table B1 for 1 in 100 as 2.33,

$$f_{cr} = \frac{5{,}000 - 500}{(1 - (2.33)(0.15))} = 6{,}917 \text{ psi}$$

The average required strength would be 6,917 psi. If the standard deviation method was used (Equation B2) and assuming a standard deviation within tests of 750 psi had been established, the required average strength would be

$$f_{cr} = 5{,}000 - 500 + (2.33)(750) = 6{,}247 \text{ psi.}$$

This is lower because of the established standard deviation of 750 psi. If testing practices were not as accurate, and the standard deviation was 1,000 psi, then the required average strength would have been 6,830 psi. Additional detailed discussion of these procedures is given in ACI 214-77 (American Concrete Institute 1984f). It should be noted that the term "quality control" entails much more than designing the concrete mix and evaluating the cylinder strength tests. The foregoing discussion of concrete strength variation should merely give an awareness that concrete having a specified compressive strength f'_c cannot be expected to provide a precisely known actual strength or other properties. Detailed specifications such as those given in ACI 214-77 (American Concrete Institute 1984f) should always be followed to ensure good quality control; however, in lieu of any better guidance, and in cases when the number of suitable tests are not available, the mixture should be designed to produce an average strength of 1,200 psi above the specified strength.

93. The above discussion relates to the design steps that should be taken to ensure proper strength of the field mixed concrete. Proper control to produce acceptable concrete in the field is achieved by the use of satisfactory materials, correct batching and mixing of these materials, and good practices in transporting, placing, curing, and testing. Although the complex nature of concrete precludes complete homogeneity, excessive variation of concrete strength signifies inadequate field quality control.

94. The proper way to ensure consistency of quality concrete is to refrain from changing any of the ingredients or steps in making the concrete. From a practical standpoint, this is almost impossible. Just obtaining cement that is consistent from bag to bag is a difficult task in addition to obtaining consistency of the aggregate, water, temperature, and a variety of other variables mentioned in Table B2. These areas should be maintained as constant as possible throughout the placement operations in order to minimize the fluctuation of the compressive strength of the test cylinders.

Table B2

Principal Sources of Strength Variation

Changes in water-cement ratio: Poor control of water Excessive variation of moisture in aggregates Retempering	Variations in characteristics and proportions of ingredients: Aggregates Cement Pozzolans Admixtures
Variations in water requirement: Aggregate grading, absorption, tion, and particle shape Cement and admixture properties Air content Delivery time and temperature	Variations in transporting, placing, and compaction: Variation in temperature and curing:

95. Aside from variations in the ingredients and the environment, the methods which are used to test the samples can vary enough to cause a discrepancy in the strength. Items such as the way the test cylinder is rodded as concrete is being placed in the cylinder mold, lack of consistency in the quality of the molds, the curing environment, the care in providing cylinder capping and testing all add to the variability of the results that are obtained from the test cylinder. All these practices should be monitored and kept constant.

96. The samples for strength tests shall be taken in accordance with ASTM C-172-82 (ASTM 1982a). The testing and the evaluation of test data shall be performed in accordance with ASTM C-39-83a (ASTM 1983a) and ACI 214-77 (American Concrete Institute 1984f).

APPENDIX C: REPLACEMENT WITH PRECAST CONCRETE PANELS

Introduction

1. The repairs of piers/wharves in a large port facility will require considerable time, particularly where extensive damage to the deck structures have occurred. It is imperative to achieve a working port environment in the shortest practicable time. While the use of cast-in-place (CIP) concreting techniques provides a more flexible port environment, the construction process is slowed by the constant building and stripping of formwork. Much of the repair of decking in a port will involve the construction of concrete spans that are repetitive in strength design and in overall dimensions. By utilizing a local precasting facility or one which is erected on site, the building and stripping of formwork are minimized. Finished panels that fit into the substructure can be assembled using a crane and a crew of workmen who can join the precast panels. The following paragraphs discuss the precasting process, what is needed to erect a precasting facility, and how to assemble one for the purposes of constructing deck panels and other structural elements for repair of war-damaged piers/wharves.

Precast concrete elements

2. Precast concrete is any concrete element that is cast in forms at a location other than its final position of use. These concrete elements may be plain or reinforced. Facilities for fabrication of precast elements are called casting plants and are either permanent manufacturing plants or temporary yards erected at the construction site. Permanent plants are usually more versatile while temporary or onsite plants are erected for a particular project.

3. The most extensive use of precasting facilities is the mass production of standardized elements. Standardization of bridge girders, piles, panels, etc. by national associations has made precasting more useful and economical. Not only are they suited for mass production of standard elements, but also precasting facilities can be adapted to construct unique and/or complex precast elements whereby position of casting, dimensional tolerances, and quality control can best be regulated to produce high quality components. This method of construction would be very well suited to repetitive beam and slab elements in the reconstruction of piers/wharves.

Onsite Precast Plants

4. The first step in setting up a precasting operation is to establish the site of the precasting plant. The choice of its location and layout can make the difference between an efficiently run operation and one that causes production bottlenecks and work stoppages. It is important to locate the plant as close as possible to the construction site. This means choosing a site behind the pier/wharf area that is remote from other port construction but is conveniently accessible to the area of pier construction so that panels can be delivered for installation by cranes. If there are several construction sites to be served by the plant, it is helpful to centrally locate the plant. It is also important to have good road access to the port rear area for resupply of materials needed in the construction of the precast elements.

Onsite precasting

5. As with permanent precasting plants, onsite precasting requires areas designated for various casting, fabrication, and storage activities. Since onsite precasting areas are usually conducted in the open, casting operations might be postponed during adverse weather unless temporary roofing (such as canvas roofing) is provided.

6. A precast plant should consist of the following minimum areas (McDonald and Liu 1978):

a. Area for concrete mixing plants.

b. Area for prestressing beds and casting other elements.

c. Areas for storage of forms and raw materials.

d. Area for storage of finished elements.

Some desirable characteristics of onsite precasting which generally are not present in permanent plant precasting include (Waddell 1974):

a. Eliminating structural elements being transported over long distances and the associated problems.

b. Transferring workers from precasting operations to construction activities as production situations change.

c. Changing production schedules as erection operations change.

Onsite layout

7. The layout of an onsite precasting plant depends largely on the size and type of precast elements for intended production. Congested or undesirable portions of the construction site should not be designated for precasting

operations. Adequate space for activities and storage has a significant impact on the speed of the casting operations. The plant should not be located to interfere with other activities, but it should have adequate area to operate effectively.

8. As areas for casting, curing, and prestressing are designated, considerations should be given to production sequences so that sequential activities are located to reduce travel distances. Also, access aisles between activities and storage areas should be adequate to allow passage of personnel as well as material and component lifting vehicles.

9. A general layout of an onsite precasting facility is composed of the following (McDonald and Liu 1978) (see also Figures C1 and C2):

- a. Two parallel concrete casting and curing areas, 20 by 200 ft each.
- b. Two stockpile areas for coarse aggregates and one stockpile area for fine aggregate.
- c. One cement silo.
- d. One 66-cu yd, 100-ton standard Corps of Engineers aggregate batching plant.
- e. One 200-barrel standard Corps of Engineers cement batching plant.
- f. Four truck mixers.
- g. One temporary storage and prestressing area.
- h. One storage area for finished product.
- i. One reinforcement and equipment shop for fabrication and storage of reinforcing bars, prestressing steel, forms, inserts, picking loops, placing, and consolidation equipment, etc.
- j. One administration building for administration office, shop engineering office, and quality control laboratory.

Flooring

10. The flooring or foundation on which precast operations are executed is an important factor in maintaining consistent standards. The flooring must be firm and level and provide proper drainage. Adequate drainage is a necessity in areas where regular cleanup involves large quantities of water. Foundations for casting beds should not have differential settlement. The flooring should be free of holes, bumps, or any obstructions which could cause accidents. These factors should be considered when choosing the proper location for the casting plant. Allocating adequate paved areas for the plant

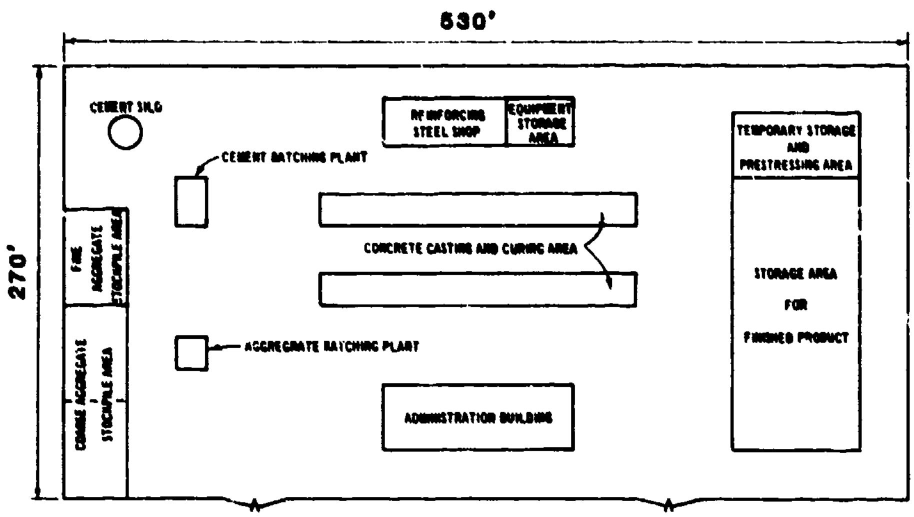

Figure C1. General plan of concrete precasting facility

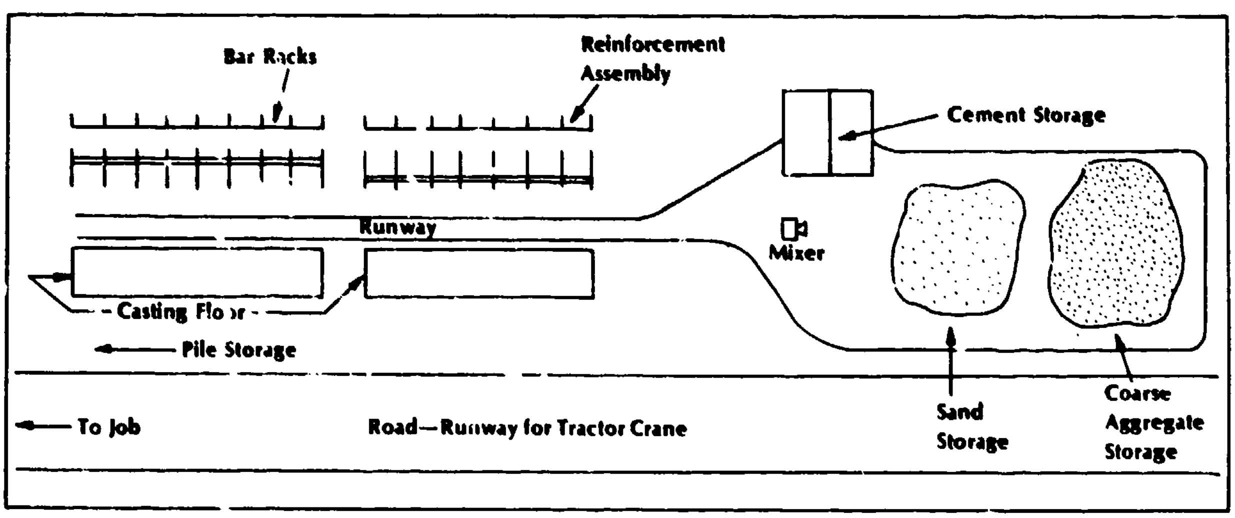

Figure C2. Typical layout of casting and material storage areas

would ensure operations to function more smoothly and cleanly. If soil surfaces must be used, proper drainage and leveling are very important.

11. Providing a layer of concrete for flooring offers several advantages in maintaining standards:

a. Workers are assured of reasonably dry conditions on which to work.

b. Sufficient anchorage can be provided for securing casting beds, work benches, etc., avoiding the delays of realignments and readjustments.

c. Smooth surfaces for cranes, buggies, etc., allow for expedient execution of handling processes.

d. Increase in drainage is accomplished by installing drainage outlets, slopes, etc.

Reinforcing steel shop

12. Each precasting plant should have a reinforcing shop located near the casting and curing area. Its purpose is to house bar stock, bending and cutting machinery, and cage fabrication along with bending and cutting operations. All steel stock should be stored in this shop to make sure that it is protected from the elements and that stocks are properly used. An area should be designated for bar cutting and bending operations and an area to house cutting and bending machinery. Bar cutting can be performed using large band saws or, if necessary, with flame cutting equipment. Flame cutting is not recommended since it changes the properties of the reinforcing bar near the flame cut. Bar bending equipment is necessary for bars larger than No. 2 bars, and the equipment is recommended on all bar bending in order to obtain uniform radii bends and excellent workmanship. The reinforcement cage construction area should be centrally located with large access doors opening onto the casting yard. The other equipment housed in the steel shop would include grinding equipment, welding and bar tying tools, and overhead hoists for lifting bulk steel and fabricated reinforcement cages.

Aggregate stockpile area

13. Both fine and coarse aggregate storage areas should be located near the concrete batch plant area. These materials must be graded and mixed prior to being batched; therefore, they should be located near the aggregate batching plant. Depending upon the casting schedule, this area will be busy. Many tons of aggregate are processed through a plant within 1 week's time. The aggregates are most likely moved by front-end loaders. Surfaced areas

adjacent to the stockpile should be provided for moving material to the batching area. Aggregate stockpiles should be separated by type and size and should be protected from the elements by roofed sheds if possible. They should also be protected from prolonged exposure to sea air which is laden with salt. Stockpile areas should also be easily accessible to roadways for resupply reasons.

Cement storage

14. Cements must be stored in a dry area since premature contact with moisture initiates the hydration process. A cement storage silo is the ideal facility. It stores bulk cement in a manner easy to access and under environmental conditions that keeps it dry. This type of closed storage is almost essential since humid air initiates cement hydration. If a silo is not available, the cement should be stored in rubber bulk cement shipping bags or be kept in delivery sacks until time to add the cement to the batching operations. A precaution is that cement dust is fine and can easily be inhaled into the lungs; thus, workers should always wear particle masks to prevent inhalation. Cement can be transported to the batch plant either by conveyor in bulk bags to be metered out at batching or in smaller bags.

Batch plant

15. The batch plant combines the raw materials before the concrete is placed in the forms. The batch plant design can range from sophisticated to very simple depending upon the resources and the size of the precasting job. All batching of materials (water, cement, and aggregates) should be proportioned by weight. Batching materials by volume leads to inconsistencies in the properties of concrete and should be avoided. For this reason, the batch plant must be equipped with good quality scales. If the concrete mixer is loaded by crane, a load cell should be installed between the delivery bucket and the crane line. The weight of the material being placed in the mixer can then be monitored from the load cell output. The batch plant should be located near the casting beds. The concrete mixer should be elevated adequately to allow the bucket or mixing truck which will transport the concrete to the casting beds to enter beneath the mixer discharge point.

Finished product storage

16. Precast units are removed from their forms long before they can be used. They must be stored at least until they have obtained sufficient strength to support 75 percent of their load capacity. This necessitates

storage yards. The storage yard size depends upon the rate of production and the size of the precast unit. The storage area should be convenient to the casting area and have access to the construction site. It should be well drained, flat, and organized to allow efficient movement of precast elements. Aisle spaces should be designed to match the material handling equipment.

17. Precast elements should be shaded if possible to prevent moisture loss from the concrete. Loss of moisture in young concrete halts the hydration process and therefore stops the concrete from gaining additional strength. Elements should be covered by tarpaulins to prevent evaporative moisture loss and if possible housed in an empty warehouse.

18. Adequate storage areas for finished elements can increase the rate of the casting operations. Storage areas for finished elements require the following considerations:

- a. Adequate support must be provided during storage to prevent introduction of unusual stresses, deflection, warping, etc.
- b. Protective guards such as wooden or rubber blocks should be provided to prevent damage when elements are stacked.
- c. Adequate access roadways are necessary to permit passage of cranes used to tow the finished elements.
- d. Storage areas for finished elements should be well organized such that the erection sequence of the elements is reflected.

Smooth plant operation

19. The smooth operation of a precasting plant depends upon adequate supplies, established production goals and methods, and coordination of personnel and materials. Listed below are several general considerations which can favorably impact precasting operations:

- a. Communication. Two-way radios should be provided so that communication is maintained between batch plant and casting personnel, equipment operators, and signaling persons, etc.
- b. Utilities. Sufficient electrical outlets should be conveniently located to minimize the lengths of electrical cords. Electrical outlets should be well protected to avoid damage from impacts and to prevent electrical shock due to dampness. Power tools and electrical equipment should use low-voltage transformers for power source. Sufficient lighting is also needed during night operations.
- c. Supplies. Adequate stocks of form oil, parting agents and fuels, must be available. These materials should be stored in suitable containers to avoid leakage or contamination. Flammable materials such as gasoline should be well marked and

stored in proper containers and storage areas as referenced in TM 5-848-2 (Department of the Army 1984c).

d. Working conditions. Working conditions should be made as safe and convenient as possible. Failure to properly consider working conditions usually result in poor workmanship, lower productivity, and workers "cutting corners" attempting to bypass bad working conditions.

Materials

20. Materials used for the production of precast concrete elements should conform to the same standards as required for in-situ production of concrete elements. Selection of cement, aggregates, admixtures, and reinforcement should be based on engineering analysis of the desired properties needed in the finished products. The produced concrete is only as good as the materials used to prepare the concrete. Therefore, in order to achieve desirable concrete elements, suitable materials must be utilized.

Portland cement

21. By definition from ASTM C 150-83a (ASTM 1983c), portland cement is a hydraulic cement produced by pulverizing clinker consisting of hydraulic calcium silicates usually containing one or more forms of calcium sulfate as an interground addition. Portland cement is the main component used to produce hardened concrete. It is available through manufacturers in types ranging from Type I through Type F. Type I portland cement is the most easily obtainable world-wide and the type probably most widely stocked in Army supply. Cements from other countries may not be typed the same as American cements. It is best for precast port construction to use a normal hydrating Type I cement.

22. As previously mentioned, portland cement should be stored in a location which is free of moisture and contamination. Cement which has been allowed to become hard and lumpy should be discarded. Additional information on sampling, testing, and types of portland cement is discussed in ASTM C 595-83 (ASTM 1983d).

Aggregates

23. The aggregate used in the production of a concrete affects material properties ranging from durability of the concrete to its modulus of elasticity. Careful choice of aggregates can improve the strength and durability of

concrete without changing the cement content of the concrete mixture. Desirable properties that are achieved by judicious choice of aggregates are discussed below.

24. Particle size distribution or "gradation" is often referred to when specifying the characteristics of the aggregates. Particle sizes of the aggregates (fine and coarse) are proportioned to provide a good bond with the cement paste. Smaller particles fill the voids between larger particles. The particle size distribution also affects the workability of the mixture, the water demand, and the strength properties. Gradation limitations are presented in ASTM C 330-82a (ASTM 1982c).

25. The aggregates used in making concrete should have proper texture and shape. The texture of the aggregates should be rough for strong bonding. Smooth or glazy textured aggregates and flat or elongated shaped aggregates should be avoided due to weak bonding results. Before adding aggregates to the concrete mixture, the aggregates should be washed to remove excess dirt, silt, and other undesirable debris. Also, the aggregates should be free of chemicals which may cause adverse reaction within the hardened concrete (i.e., chemically reactive aggregates may cause corrosion of reinforcement with the concrete elements). Refer to ACI 221R-61 (ACI 1984h) for additional insight into choices of aggregate.

Admixtures

26. Admixtures are materials, other than water, cement, or aggregates, which are used to permanently or temporarily change the properties of the concrete. One method of classifying chemical admixtures as established by ACI 212.2R-81 (ACI 1984c) is based on the function of the admixture:

a. Type A, Water-reducing admixtures.

b. Type B, Retarding admixtures.

c. Type C, Accelerating admixtures.

d. Type D, Water-reducing and retarding admixtures.

e. Type E, Water-reducing and accelerating admixtures.

When properly used, admixtures can induce desirable properties in the concrete mixture such as workability, air-entrainment, reduction in cement/water ratio, accelerated strength, heat reduction, control bleeding, etc. However, just as cement, water, and aggregates must be properly proportioned in the concrete mixture, admixtures must also be carefully proportioned in order to serve

their intended purposes. Usually the addition of an admixture requires the reduction of another material in the concrete mixture.

27. There are numerous varieties of admixtures presently in use, and many are discussed in detail in various ACI and ASTM publications. The use of these admixtures should follow the guidelines established by organizations such as ACI and ASTM. A good reference source which discusses the use of admixtures is ACI 212.2R-81 (ACI 1984c).

Mixing water

28. The purpose of adding water to concrete is to trigger the hydration process in the cement and to lubricate the mixture so it consolidates properly. As a general rule, any natural water acceptable for human consumption without suspect odor or taste may be used for making concrete. However, water which is not safe to drink can be used to make concrete. While some impurities in the mixing water are acceptable in reasonable amounts, excessive impurities may adversely affect setting time, strength, and durability of the concrete. Some impurities (i.e., salts of manganese, tin, and lead) can cause adverse reactions if only very small percentages are contained in the mixing water. Therefore, it is good practice to prepare test cylinders using water of known good quality and using the available water to make strength comparisons. Algae contained in mixing water should be avoided. Algae attaches itself to the cement and reduces the cement to aggregate bond. The result is a significant reduction in the strength of the concrete.

Seawater

29. Seawater should be avoided for making prestressed concrete because of possible corrosion activity between the saltwater and the steel. The use of seawater can also increase the chance of corrosion in conventional reinforced concrete. It is acceptable for use in unreinforced concrete where reactivity is not a problem. When using muddy water, it should be allowed to settle before adding to the mixture to reduce the silt and clay added to the concrete.

Forms

30. Similar material employed for formwork or "casting beds" of in situ concrete casting can be used in precasting. The more common materials are wood, concrete, and steel. Recently, the introduction of glass-reinforced plastics has proven successful for formwork. The forms should remain level and aligned throughout repeated uses and should be set at a height best suited

for workers. Enough forms should be provided to avoid extended delays between casting and stripping of forms.

31. Forms should be able to resist warping or other distortions which adversely affect the accuracy of the finished element. Joints should be tight to prevent leakage. Cleaning should be accomplished following each casting with special attention given to holes for inserts and joints. Forms should also be protected against frost. Measures should be taken to prevent water from accumulating within forms during heavy rainfalls.

Types of forms

32. The type of forms that are chosen will depend upon the size of the precasting operation and the type of elements to be fabricated. Large precasting operations, where simple shaped elements are to be cast in large numbers would dictate steel forms while small operations may require wood forms. The following discussion relates the different types of forms to their major uses and economies.

33. Wood forms. Wood in the form of solid material and plywood sheets has long been used in abundance for forms with a high degree of success. The necessary skilled workers are available in most situations to construct wood forms which meet production specifications. Wood forms for pile construction are illustrated in Figure C3. Wood forms are the most economical of all the forms. They are easy to construct, reconstruct, and change without much difficulty. Wood forms are not as durable as other formwork materials and will show signs of wear after a few repeated uses. If a large number of units of the same type of precast element are needed, wood forms should not be used. Steel will ultimately prove more economical.

34. Steel forms. Steel forms are used when repetitious element fabrication is required. Steel can be scavenged from available structural steelwork and reworked into the shape of the forms. Steel forms can be fabricated in sections to reduce erection and stripping time. Stack forms (forms designed to allow more than one similar element to be cast at the same time) are normally constructed of steel. Because steel is a rigid material, panels made from steel forms are truer to their original dimensions than those made from wood forms, and there are less chances that mortar will leak from steel forms due to bowing of the formwork. Steel costs more than wood, but steel lasts longer and is recommended for large precasting jobs.

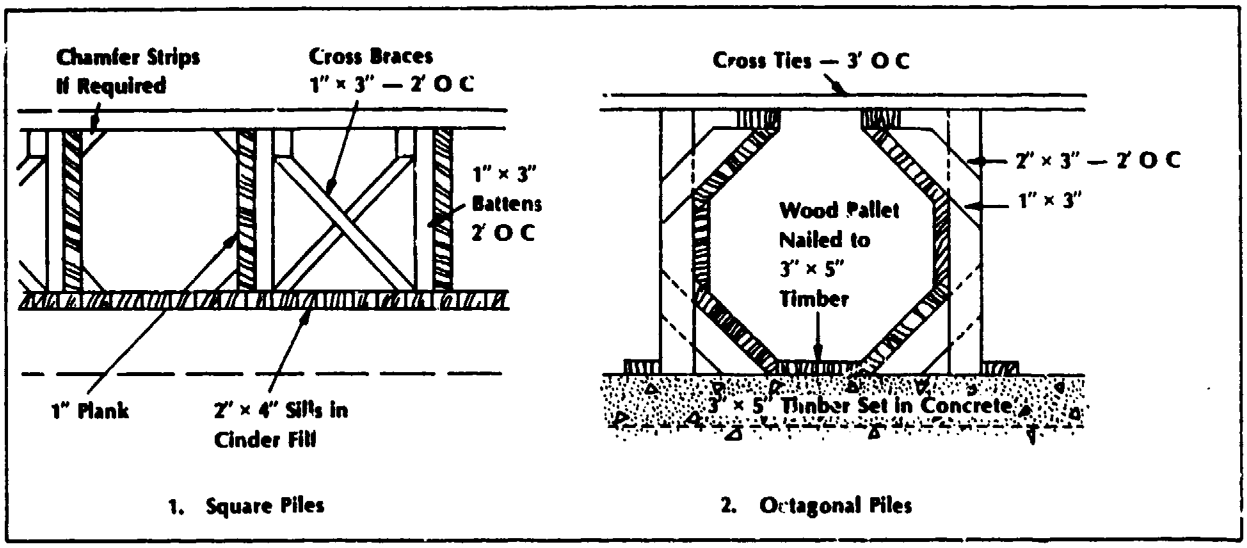

Figure C3. Detail of wooden forms for square and octagonal pile construction

35. Glass reinforced plastics. Recently, the introduction of glass reinforced plastics for constructing forms has proven very promising. If the plastic material is sufficiently stiffened to resist site handling and other stresses, these materials produce forms with very durable faces. Glass reinforced plastics are factory manufactured only and must be transported to the construction site. This makes them unsuitable for military site work; however, their acceptance as the major forms for precast one- and two-way slab construction (see formwork discussion starting with paragraph 25 in Appendix B) would make their inclusion in the Army inventory of construction materials highly desirable.

36. Concrete forms. Concrete forms are extremely useful for onsite precasting. Concrete provides the durability needed to resist weathering and the impact forces of finished elements striking the forms during removal. The use of concrete forms allows the normal skills of the concrete workers to be utilized in the construction of the forms. Wooden forms would be used to cast the concrete forms, and then the wood can be discarded and the concrete used to cast other concrete members. With concrete forms, care must be taken in moving the forms and preventing bonding between the new and old concrete.

Precast Elements for Piers/Wharves

Port facilities

37. The construction of port facilities include such structures as piers/wharves, fender systems, breakwaters, storage, utilities, and administrative facilities. When properly planned and designed, precast elements can comprise a major portion of the components used to construct such structures as those mentioned. Particular elements including piles, pile caps, deck slabs, caissons, girders, and panels are examples of structural elements utilized in port construction for which onsite precasting can be adapted. Listed below is a brief description of some structures employed for port construction. A more detailed description of these and other structures used for port construction is described in TM 5-850-1 (Department of the Army 1984b).

38. Piers/wharves. Piers/wharves are used to permit transfer of passengers and cargo from sea to land and vice-versa. These facilities also provide docking space for transporting vessels. Particular precast elements that are useable include piles, pile caps, deck slabs, girders, beams, and caissons.

39. Fender systems. Fender systems provide protection to transporting vessels and the dock from damages which could result from abrasive and impact forces when contact between the two occurs. The recent introduction of prestressed concrete fender piles with rubber buffers attached at the ends has proven successful (Figure C4). Fender systems are normally attached to the pier/wharf structure and can also be a retractable absorptive system.

40. Miscellaneous facilities. Facilities for storage, utilities, and administration provide shelter for cargo, equipment, and personnel. Storage facilities such as "transit sheds" are often a part of the wharf structure. These types of facilities can also be placed at convenient locations elsewhere in the port area. Particular precast elements include panels, beams, slabs, and girders.

Piles

41. The usual function of piles is to transmit foundation loads to stable soils in an attempt to avoid differential settlement within the foundation. Precast concrete piles with reinforcement are available in a variety of shapes (round, square, and octagon) and lengths. Lengths of up to 50 ft are common for normally steel reinforced piles. Because of the added strength and

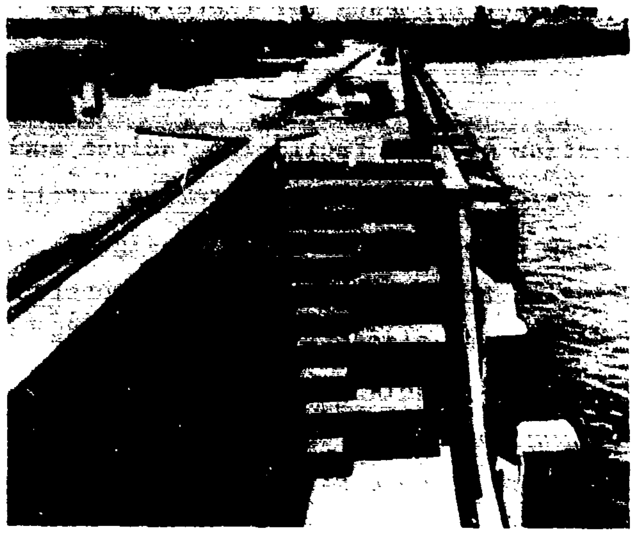

Figure C4. Precast concrete fender piles (Courtesy of Van Nostrand Reinhold Company, Inc.)

durability, longer lengths are available with prestressed piles. Hollow core piles should be given consideration when the ratio of the piles unsupported length to cross-sectional area is large (McDonald and Liu 1978). Reinforcement and/or prestressing of piles should provide adequate strength and durability to withstand handling stresses, structural loads, and the driving forces needed to embed the piles to the required depth. Because of the size and weight of precast concrete piles, the layout of the casting yard should be planned to minimize travel distance from pile storage to the construction site. Figure C5 shows a typical detail of solid octagonal and square precast piles.

Sheet piles

42. Precast concrete sheet pile elements are normally driven in the ground side by side to form a continuous wall. Sheet piles differ from the previously mentioned piles in that they are not bearing piles, but retaining piles described in FM 5-134 (Headquarters, Department of the Army 1985b). Several techniques are used to connect the piles including tongue and groove interlocks, welding with grout seal, and embedding steel elements in the joints of the precast concrete piles (to increase resistance to tension). As with practically all precast concrete elements, prestressing will increase strength and durability. Figure C6 shows several cross-sections of concrete sheet piles.

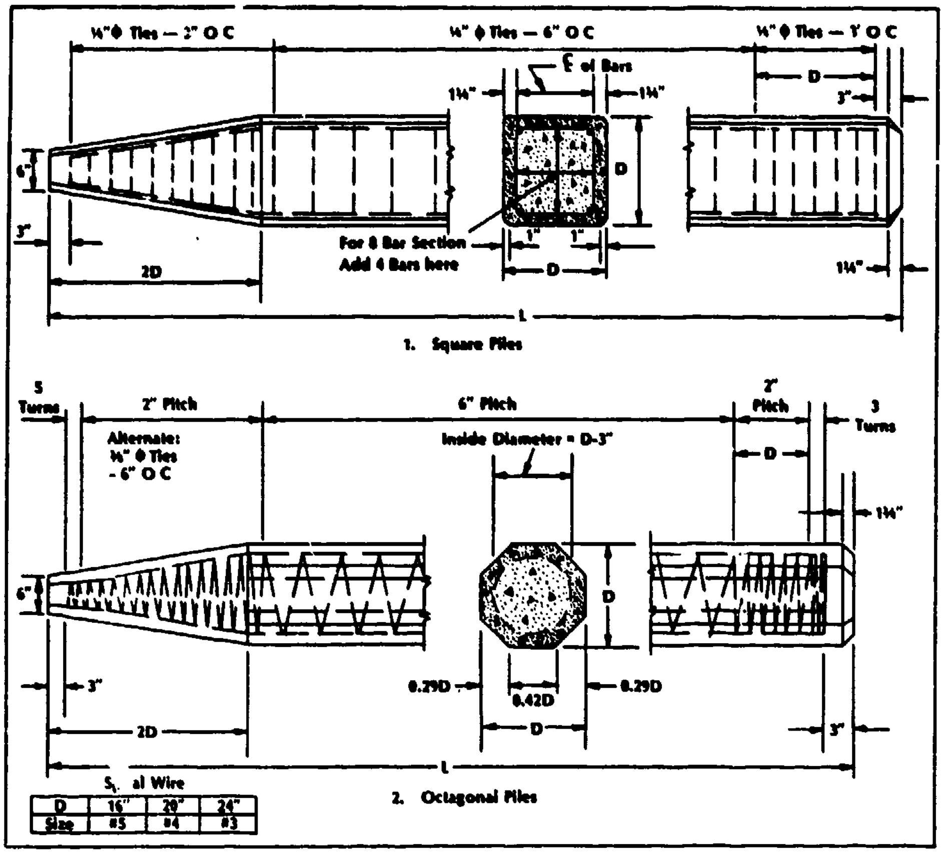

Figure C5. Typical detailing of pile elements

Decking elements

43. Precast concrete elements included in this group are pile caps, decking slabs, edge beams, curbs, etc. Deck slabs may be either solid or hollow-cored (Figure C7). Greater spans are permitted when hollow-cored slabs are used. Deck slabs should be designed for continuity. Proper alignment of the precast slab elements can be maintained by using transverse ties to join the elements together. Prestressing allows greater spans, and tolerances can be achieved.

44. Pile caps are used to distribute the forces from the superstructure to the bearing piles. When precast concrete pile caps are used as separate elements, driving forces must be withstood. Connection of the pile caps to the bearing piles must assure proper transmittal of loads. It is also a good practice to design pile caps slightly oversized in width and length to compensate for alignment tolerances with the piles. A more efficient method is to include pile caps in the decking slab design as one continuous element which

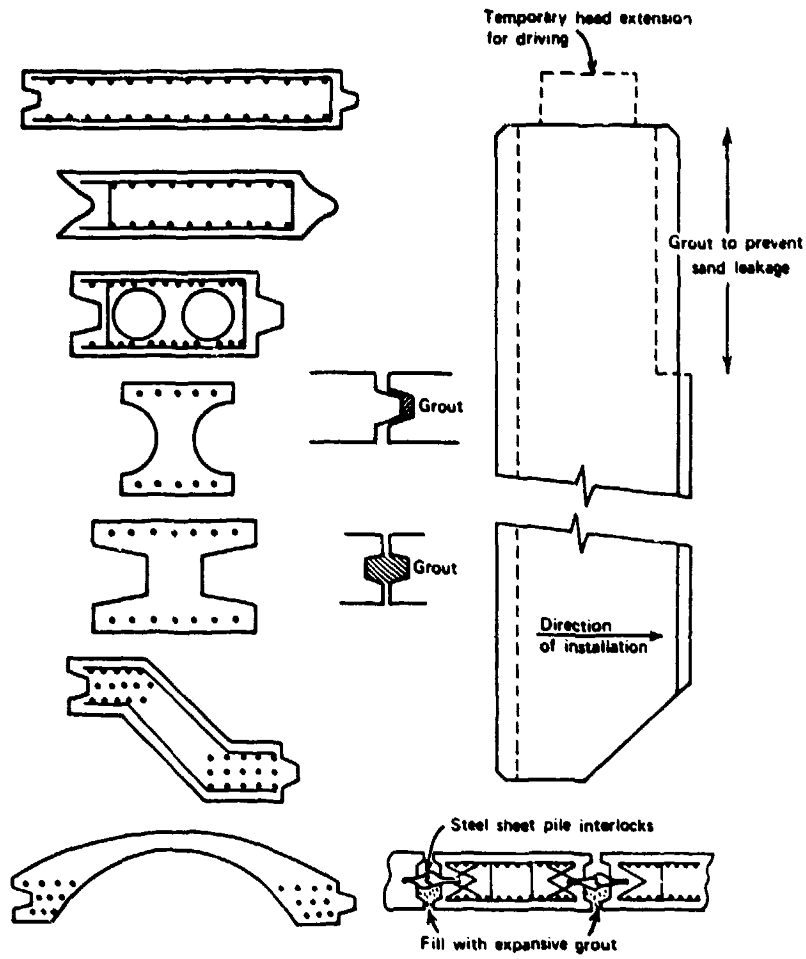

Figure C6. Typical detailing and cross-section of concrete sheet pile

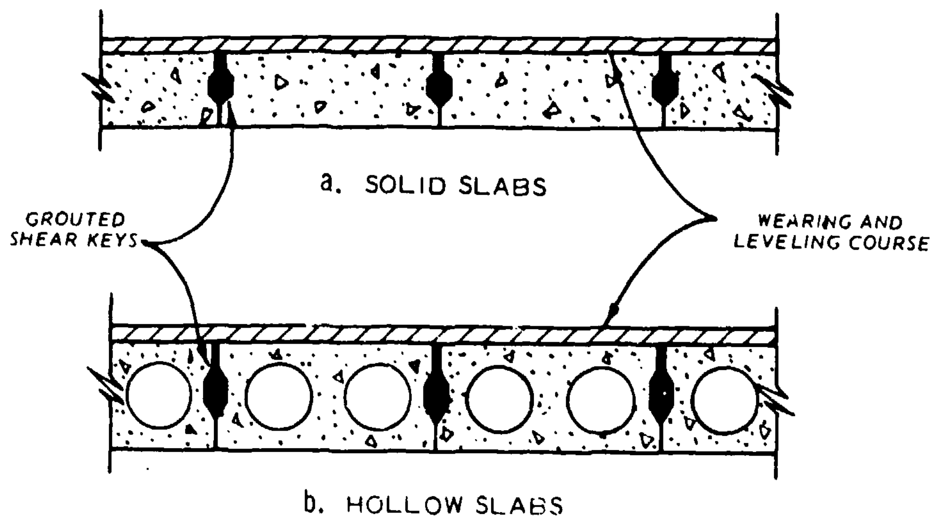

Figure C7. Typical sections of decking slabs

helps to ensure continuity of the pile caps. Formwork for this type design is discussed in paragraphs 25 through 96 of Appendix B.

45. Precast concrete edge beams, catwalks, and curbs comprise the remaining elements for deck structures. These elements can also be included in deck slab design or can be precast as separate elements.

Panels

46. Precast concrete panels can be used to form the walls and roofs of storage and administrative facilities. Openings for doors, and windows can be boxed out in the formwork. Strongbacks or braces should be used to prevent damage during handling of panels with irregular shapes and/or numerous openings.

Miscellaneous elements

47. Other precast concrete elements used in port construction such as caissons, block walls, girders, and monoliths provide the stability and durability necessary for use in this type of environment. Proper design of concrete mixture proportions and consistent quality control measures are the key to precast concrete elements realizing their potential in port construction.

Production Principles

48. The adoption of thoroughly planned production principles is needed to achieve the maximum production within a given onsite precast operation. Planned production and skilled stock management are key factors in determining the quantity and quality of production output. Material flow, handling techniques, and storage practices are particular examples.

49. Arrangements for stripping and storage of finished elements to be conducted on alternating shifts allow casting operations to take place during peak activity hours. As mentioned earlier, related areas of activities should be located to avoid congestion and minimize travel distances. Also adequate storage areas are important in controlling production flow.

50. The use of accelerated curing reduces the time stripping and handling can be accomplished. The use of conveyors, buggies, and forklift trucks, wherever possible, rather than cranes for handling material and finished elements requires fewer skilled personnel and specialized equipment.

Personnel

51. The presence of a civil or structural engineer familiar with concrete technology is mandatory to control overall operations. A sound management team is needed to establish, coordinate, and maintain production output. Appropriate personnel within the engineer group can be easily trained to perform compaction, vibrating, placement, and insert embedding techniques as the job progresses. Carpenters, welders, steelworkers, and equipment operators are examples of well-trained skilled workers who are necessary for formwork construction and materials handling.

Curing

52. Curing is the process of maintaining the moisture and temperature levels of newly cast concrete elements such that the process of hydration can continue to develop desired characteristics within the concrete. Proper curing of concrete elements is essential to developing good quality concrete. Properties such as strength and durability are dependent on the curing process. There are basically two methods of curing concrete elements, moisture curing and chemical curing.

53. Moisture curing. Techniques used in moisture curing of concrete elements include ponding, flooding, sprinkling, and the use of moisture retaining materials. Moisture curing assures that an adequate amount of moisture is always present for hydration within the concrete. Ponding involves completely immersing elements in water usually through the use of tanks or impoundment dikes. Sprinkler systems use the continuous or intermittent spraying of water on the concrete. When using sprinkler systems for curing, the wetting interval must ensure that drying of the concrete is not permitted. Moisture retaining materials such as burlap, rugs, or cotton mats are used in curing concrete by soaking the material and completely covering the elements with the material. A sheet of plastic or another type of waterproofing material can be placed over the soaked absorbent material to help retain the moisture longer.

54. Chemical curing. Chemically curing concrete elements involves applying a sealing compound over the concrete to form a membrane which retards or eliminates evaporative moisture loss through the membrane. Compounds such as waxes, resins, chlorinated rubbers, and other solvents are used in chemical curing. The sealant compound should be uniformly applied to the concrete as soon as finishing has been accomplished. If drying occurs prior to

application of a curing compound, the concrete should be "wetted down" to uniform dampness before applying the compound. Also, caution should be taken not to use compounds on concrete surfaces which will be required to bond with concrete placed at a later date or surfaces which are to be painted unless proper measures for removing the compound material have been established.

55. The curing cycle of precast concrete elements should be standardized so that each element receives the same curing procedure. ACI 308-81 (ACI 1984j) requires a 3- to 7-day curing period prior to stripping and/or handling of concrete elements cured by normal curing methods. However, such waiting periods would mean significant delays in construction activities. As a result, various methods of accelerated curing are used to hasten stripping and handling of newly cast elements.

Accelerated curing

56. In order to minimize the time required before concrete elements attain sufficient strength for stripping and handling, accelerated curing techniques are employed. The use of electricity or steam makes accelerated curing possible. Since the rate of hydration (chemical reaction process from which concrete hardens and acquires strength) increases with increase in temperature, accelerated curing is achieved by immersing the newly cast elements into a steam bath which elevates its temperature. By elevating the ambient temperature around the precast element to a level of 140 to 150°F for 18 to 24 hr the element attains sufficient strength to be stripped at the end of the 24 hr. Also, accelerated curing reduces storage space since elements can be handled and erected in a shorter cycle. Steam curing, while increasing efficiency of precasting plants, is very energy consumptive. It is unlikely that this curing technique will be used in port repair scenarios.

Casting and consolidation

57. Casting and consolidation procedures used in the production of precast concrete elements should be clearly understood by all personnel involved. The goal is to eliminate wasteful practices and to produce quality concrete. Improper casting practices and poor consolidation result in air voids, segregation, strength reduction, and other undesirable characteristics in the finished elements.

58. Dropping fresh concrete into the forms from excessive heights should be avoided. Separation of materials upon impact may result. Fresh concrete should be placed as close to its final position as possible.

Allowing fresh concrete to "run" long distances, as is sometimes performed by vibrating the concrete until it flows from one location to another, can result in honeycombing and segregation as the cement paste flows ahead of the coarser materials. If the fresh concrete is to be placed in layers, 12- to 18-in. layers should be placed and properly consolidated before placement of subsequent layers. Wood forms should be moistened prior to casting to avoid removing moisture from the fresh concrete.

59. The most commonly used method for consolidation of fresh concrete is vibration. Mechanical vibratory equipment is available to provide either external or internal vibration (Figures C8 and C9). External vibrators can be attached to casting forms. Vibrating tables are also used for external vibration. Although more suitable for smaller elements, two or more vibrating tables can be employed for larger precast concrete elements. Care should be taken to assure uniform vibration especially between the vibration tables. Internal vibrators are immersion-type tubelike devices which use offset cams to produce vibration when immersed in the concrete. These devices should be inserted perpendicular to the free surface of the concrete and never used to horizontally move concrete from one location to another. Over vibration should be avoided. This will cause entrained air bubbles to bleed from the concrete and reduce the overall durability of the concrete mixture.

Prestressing

60. Prestressed concrete is formed by placing either wire strands, tendons, bars, or steel cables in areas within the concrete element where expected high tensile stresses are developed. Counter stresses are then induced through these strands to offset the tensile stresses which result from design loads and handling. Prestressing concrete causes the entire cross-sectional area to contribute in resisting imposed loadings. Precasting operations provide a convenient and readily adaptable setting for prestressing techniques and can be applied to virtually all types of structural applications.

61. The two methods of prestressing concrete elements are pretensioning and postensioning. Pretensioning involves prestressing the in-place strands in the formwork prior to casting of the fresh concrete. Hydraulic type jacks are commonly used to provide loading for prestressing. After the concrete has hardened and bonded with the pretensioning steel, the strands are released and compressive forces are transmitted to the concrete. Postensioning involves

Figure C8. External form vibrators used in precast beam construction (Courtesy of American Concrete Institute)

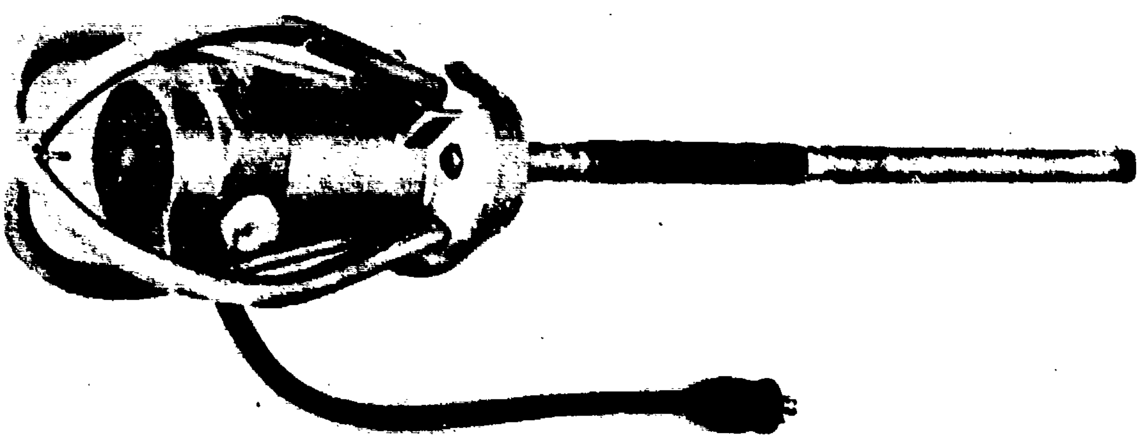

Figure C9. Typical commonly used internal post type vibrator (Courtesy of American Concrete Institute)

stressing of the strands after the concrete has hardened. The unstressed wires are placed in conduits which are cast into the concrete element. After the concrete has hardened, the wires are tensioned. The free ends of the wires are mechanically anchored to the ends of the member which allows transfer of tensile forces from the steel to the concrete (Figure C10).

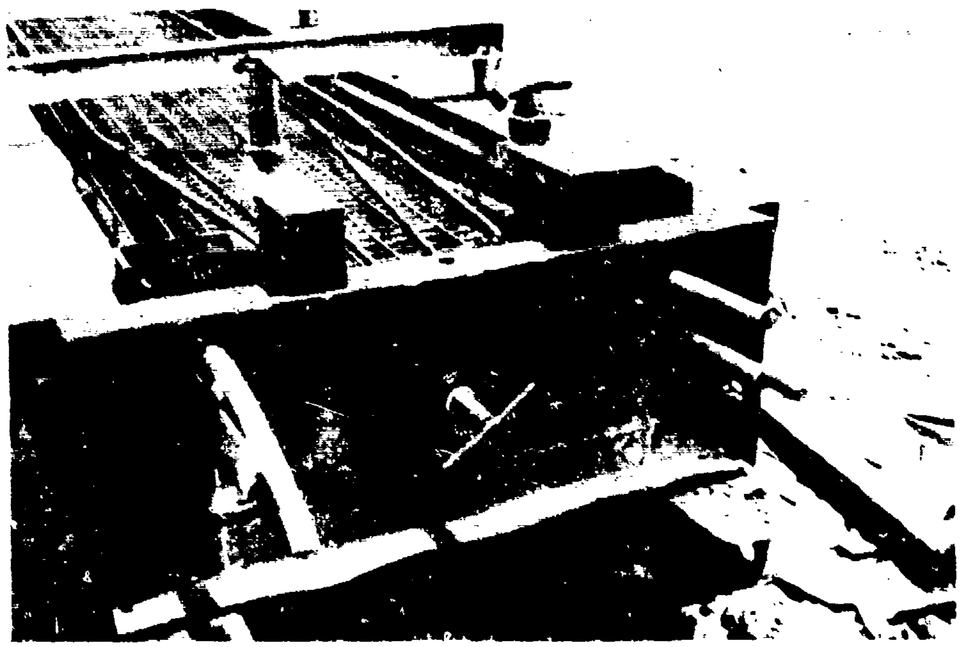

Figure C10. End view of postensioning setup for precast concrete girder

62. Prestressed concrete elements result in highly durable, crack-free, and stronger elements. When high strength materials are used for prestressing; longer, lighter, and smaller elements may be specified compared with ordinary reinforced elements. During the prestressing operations, personnel should be kept clear of the stressing area in case of slippage or breakage of the strands. Details of the prestressing operations for theater of operations casting yards are discussed in Technical Report C-78-1 (McDonald and Liu 1978).

Handling Precast Elements

63. Handling precast concrete elements begins with removal from casting forms and terminates with erection at the construction site. Handling techniques for moving precast elements are an important aspect of the overall

precasting operations. These elements are often subjected to unusual stresses during handling when they have gained very little strength. Proper application of handling techniques requires careful considerations.

64. When planning an onsite precasting operation, there is no standard system for handling precast elements. A good handling system is considered one which is suitable for a particular precasting operation and which provides safe and efficient methods of moving the members without damage. Handling operations should be made simple, wherever possible, so that even semiskilled workers can perform the handling procedures. Smooth flow of finished elements, minimum travel distances, and reduction in the effort required by workers are characteristics of proper handling operations.

Damage control and safety

65. The main consideration when moving precast members to installation sites is to make sure damage to the elements is avoided. To assist in damage control; proper design, selection, and inspection of all lifting attachments, arrangements, and equipment are needed. The greater the number of times the precast elements are handled, the greater the potential of damage. Therefore, a well conceived handling system should be implemented to minimize the handling of precast elements.

66. As precast elements are moved, measures should be taken to protect the elements. When slings are involved, rubber or wooden guards should be used between the slings and the concrete to avoid breakage of the corners and edges. For inserts, anchors, and other lifting hardware, only ductile steel should be used. Brittle failures provide dangerous situations if high strength steels are stressed to yield limits. Worn or damaged lifting devices and cables should be avoided for handling precast elements.

67. Any system devised for handling precast elements should include personnel safety as an integral part of the handling procedures. In most cases, damage control and safety measures are interchangeable as measures taken to prevent damage and serve as a safety precaution and vice-versa. Assuring the elements are handled only by preplanned procedures reduces hazards and chances of damage.

Handling stresses

68. Stresses experienced by precast elements during handling operations often exceed stresses computed for the structural design of the precast elements. As a result, handling stresses may dictate the size and selection of

the precast members. Handling stresses also include stresses imposed on lifting attachments in addition to the precast elements.

69. The usual placement of reinforcement in the tensile zone may not compensate for flexural stresses developed during lifting. It should therefore be the goal of the designed lifting arrangements to maintain these stresses below the tensile capacity of the concrete by properly locating pick-up points. Another alternative is to provide additional reinforcement in the regions where lifting stresses are most severe.

70. When precast members are being handled, the stress distributions throughout the element should be analyzed for every position during lifting. Severe and unusual stresses may develop as lifting cables and the members form various angles with respect to each other. In this situation, horizontal and vertical loads can develop causing bending stresses on attachments which may cause localized crushing of the concrete. Spreader bars or lifting beams are required to eliminate the horizontal forces.

71. In situations where flat members (i.e., panels) are raised to a vertical position, anchor attachments should be designed for shear forces. Although tensile stresses are large in magnitude during initial lifting, shear stresses predominate as the member approaches a vertical position. Anchor attachments designed for shear require larger sized anchors.

72. Analysis of handling stresses should be determined during the initial design stages of the precast elements. In this manner, size and selection of the elements and handling attachments can be designed with adequate safety factors to compensate for unusual handling stresses. This also permits specifying similar members and attachments which could serve to simplify the overall precasting operations.

Lifting arrangements and hardware

73. One of the primary considerations influencing the designation of lifting arrangements for precast members is the size and weight of the members. Locating the center of gravity usually provides the most balanced position for attachments and loads. Pick-up points are critical and members should be handled only by the designed pick-up points. Designation of pick-ups should reduce and control the location and type of stresses incurred. There are numerous lifting techniques in practice which enable efficient handling of precast elements. The ACI publication by Waddell (1974) provides

useful equations for determining lifting arrangements for precast panels based on locating the center of gravity.

74. Any lifting device consists of two parts (Waddell 1974).

a. The anchorage element embedded into the precast concrete.

b. The attachment elements attached to the anchorage to fasten lifting lines to the unit.

Regardless of anchorage device type, sufficient depth must be provided for pullout resistance, and the concrete must reach the desired strength before imposing lifting loads on the anchorage device.

75. Slings. Slings provide practical lifting arrangements, and the need for any type insert is eliminated. In this respect the use of slings is advantageous. Disadvantages of slings include provisions needed when stacking members and the introduction of unusual stresses within the members. Wide slings are preferred to aid in avoiding damage to the slings and precast members.

76. Loops. Loops of steel embedded to sufficient depths in precast elements permit a simple arrangement for lifting attachments with enough of the loop remaining exposed for attachment to a crane hook. Loops can be placed in the elements at an angle appropriate to the anticipated lifting direction. This avoids bending of the steel loops which could result in undesirable angular forces. To provide maximum pullout resistance, the loops should be anchored against the normal reinforcement.

77. Spreader bars. In addition to eliminating angular forces on anchorage devices when lifting cables are used, spreader bars or lifting beams allow multiple lifting points to be utilized. This is useful in reducing bending stresses within elements during lifting. Spreader bars also provide convenient lifting arrangements for handling large structural components with irregular geometry, recesses, reduction in sections, and elements which are long and slender. Allowances should be made for variations in the center of gravity.

78. Insert anchors. There are numerous insert anchors available from manufacturers to adapt almost any conceivable need for lifting precast elements. The most commonly used are the single cast-in-socket types. Insert anchors are made from cast material, machined steel, or plastic materials. Insert anchors should be protected against clogging during casting activities.

They must also be embedded to a depth necessary to assure that the shear cone, normally anticipated with pullout, has enough area to resist handling loads.

79. Other available types of insert anchors include lifting eyes, threaded bolts, and numerous variations for particular applications. Insert anchors are available which automatically locks the lifting eye to the embedded stud. Because these anchors must be released by hand, accidental loosening is prevented. Another form of anchorage attachment is screw anchorage in which spiral threads on anchor screws form a threaded recess in the concrete at the time of casting. Upon removal, the remaining threaded recess allows screws or bolts to be reinserted for lifting attachment. As a result, the need for permanently placed inserts is eliminated. Plated and shoed insert anchors of various lengths are available ensuring adequate embeddment and bonding. Illustrations of various lifting arrangements and hardware are shown in Figures C11 through C16.

Handling equipment

80. There are many types of equipment available for handling precast elements. Since each type offers various advantages and disadvantages, it is necessary to determine which equipment is most suitable for a particular onsite layout. Among the various choices are cranes, forklifts, vacuum lifting devices, straddle carriers, and small hydraulic cranes which can be mounted on trucks. Manufacturers' recommendations for load capacity limitations and service requirements should be followed for equipment selected.

81. Straddle carriers offer several advantages including 90° pivoting, large width and height clearances, and high loading capacities. However, justification for utilization of straddle carriers is usually limited to permanent manufacturing plants or large onsite operations. Forklifts are convenient and useful for handling precast elements (Figure C17). There are a number of forklift trucks in the military inventory including the container handling forklifts which can be used for this purpose. Wide frames can be attached to forklifts to allow handling of longer elements. Small hydraulic cranes mounted on trucks such as the one shown in Figure C18 are also very convenient for onsite handling of precast elements. This equipment is equipped with booms and can also be used in erecting the elements at the construction site. Considerations should also be given to handling equipment which use batteries for power. Satisfactory service can be provided by

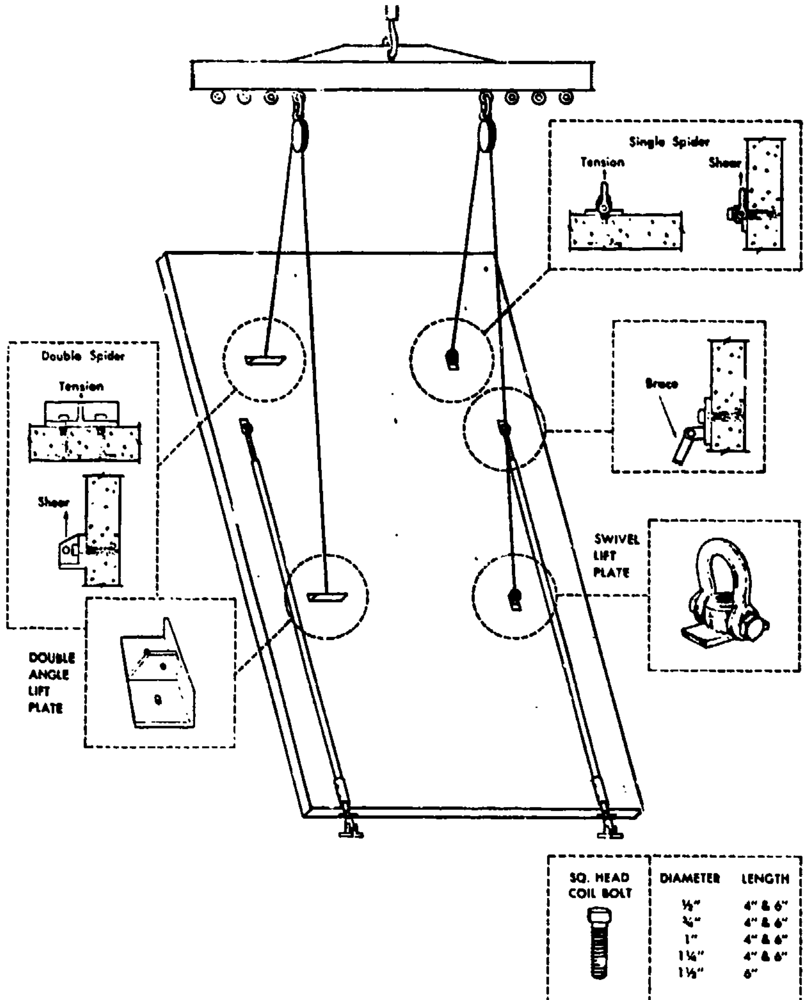

Figure C11. Lifting arrangement for precast panels (Courtesy of American Concrete Institute)

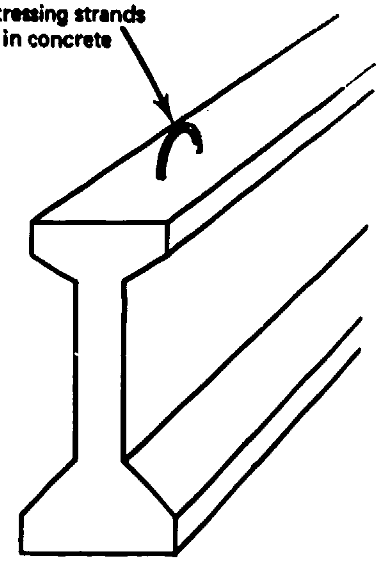

Figure C12. Scrap prestressing strands embedded to provide lifting eye (Courtesy of American Concrete Institute)

battery powered equipment, and exhaust fumes can be eliminated from gas powered equipment.

Transporting precast elements

82. As mentioned earlier, one of the advantages of onsite precasting is the elimination of long distance hauling of elements. This allows employment of similar equipment used in handling the elements as well as being utilized in transporting and erection. Still, short distance transporting of the elements to construction sites might be required and general transporting considerations are needed. The vehicle used to transport the elements should be a size and capacity that is not exceeded. Transport routes should be free of bumps and other obstructions to reduce impact and vibration forces which could prove damaging to the structural integrity of the elements. Adequate tie-down security is needed to avoid movement of the elements during transport. Protection should be provided between elements to provide support and to avoid damage. Loading of elements should also provide the most balanced positioning with respect to the center of gravity of the transport vehicle. Finally, the vehicle operator should always have a clear view of the area and the elements enroute to the construction site.

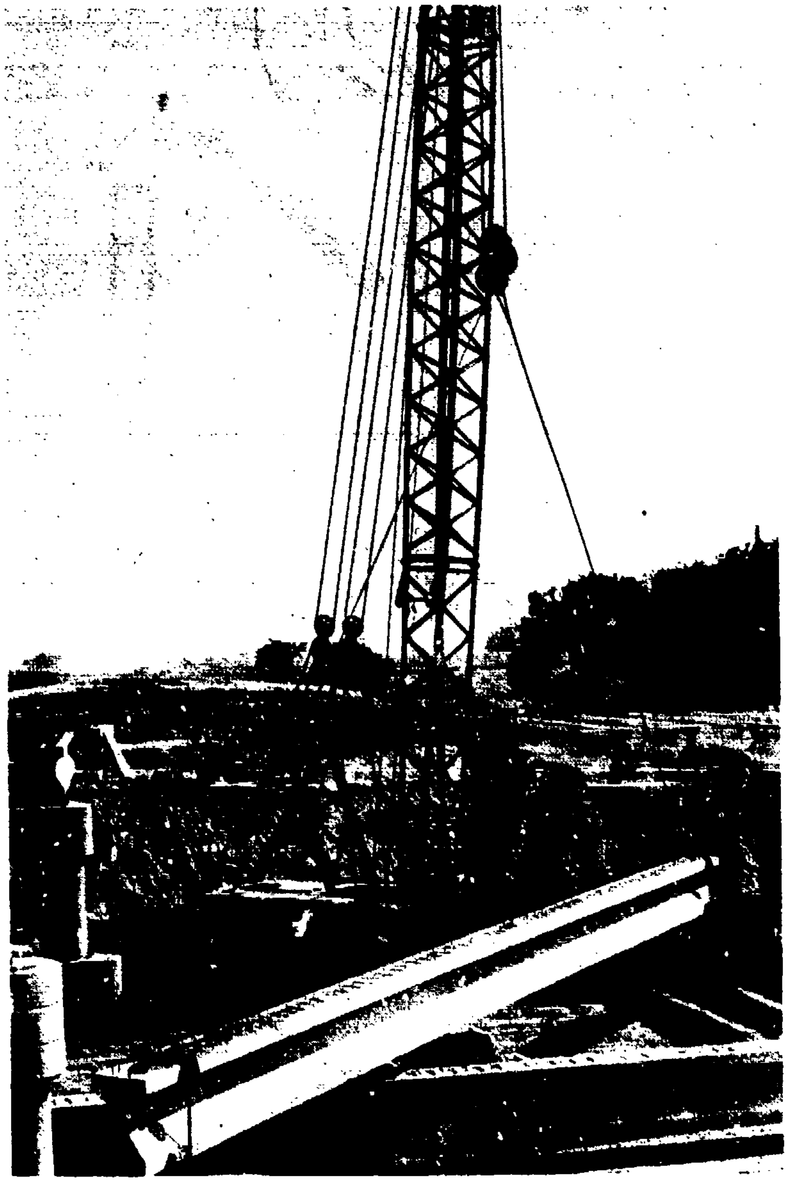

Figure C13. Girder lifted by slings attached to inserts at ends (Courtesy of American Concrete Institute)

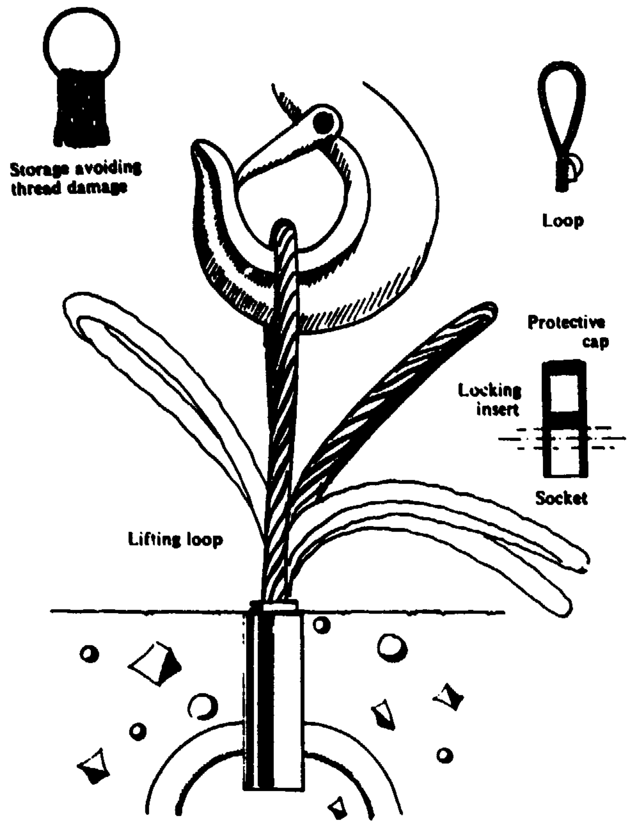

Figure C14. Embedded loop providing lifting attachment (Reprinted from "Precast Concrete Production" by J. G. Richardson)

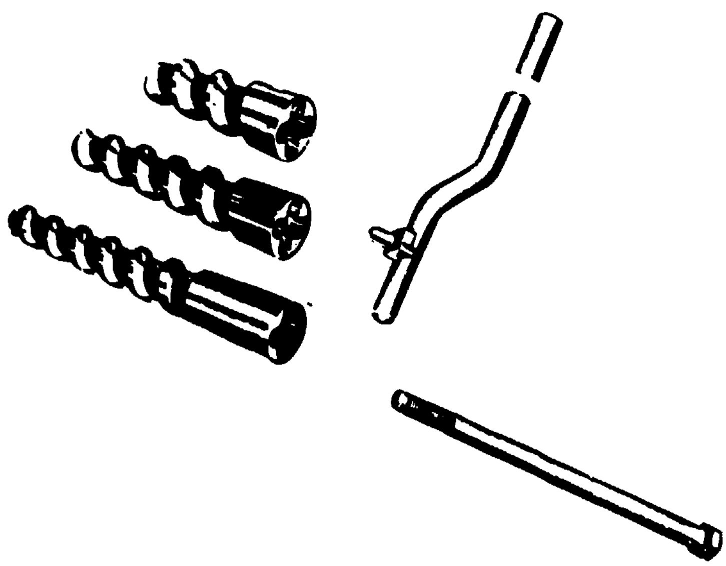

Figure C15. Screw anchorage (Reprinted from "Precast Concrete Production" by J. G. Richardson)

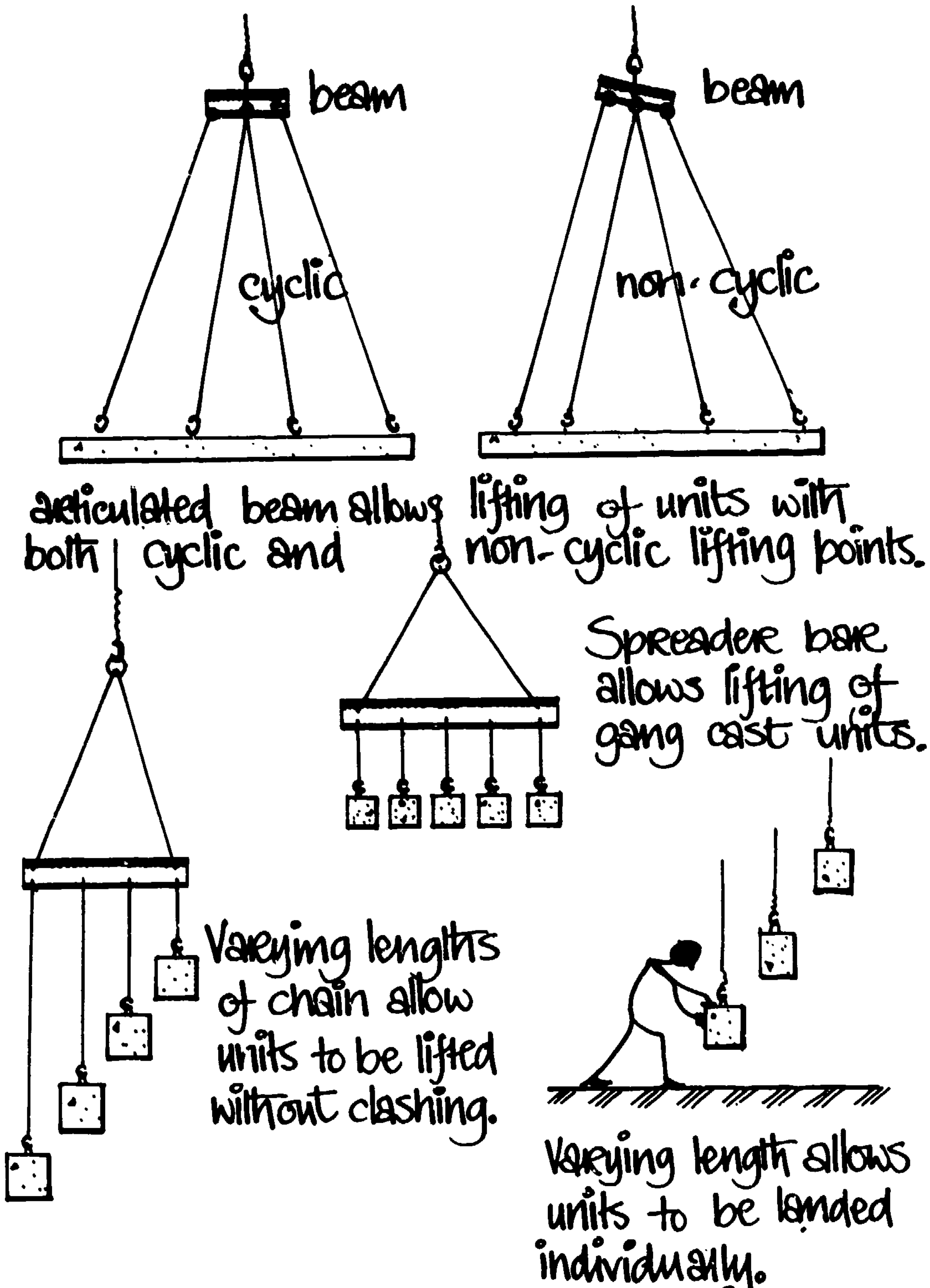

Figure C16. Spreader bars providing various lifting arrangements (Reprinted from "Precast Concrete Production" by J. G. Richardson)

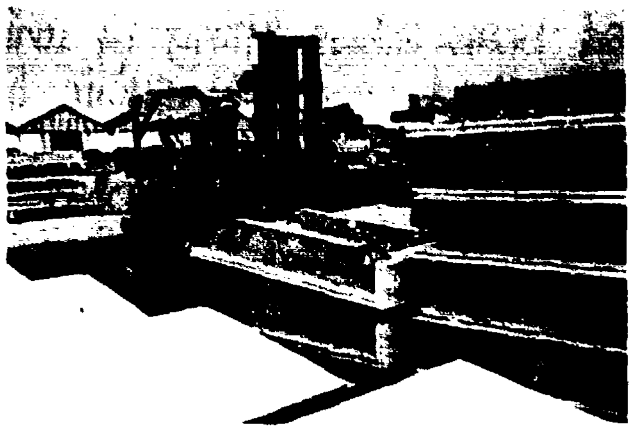

Figure C17. The use of forklift in precasting yard (Reprinted from "Precast Concrete Production" by J. G. Richardson)

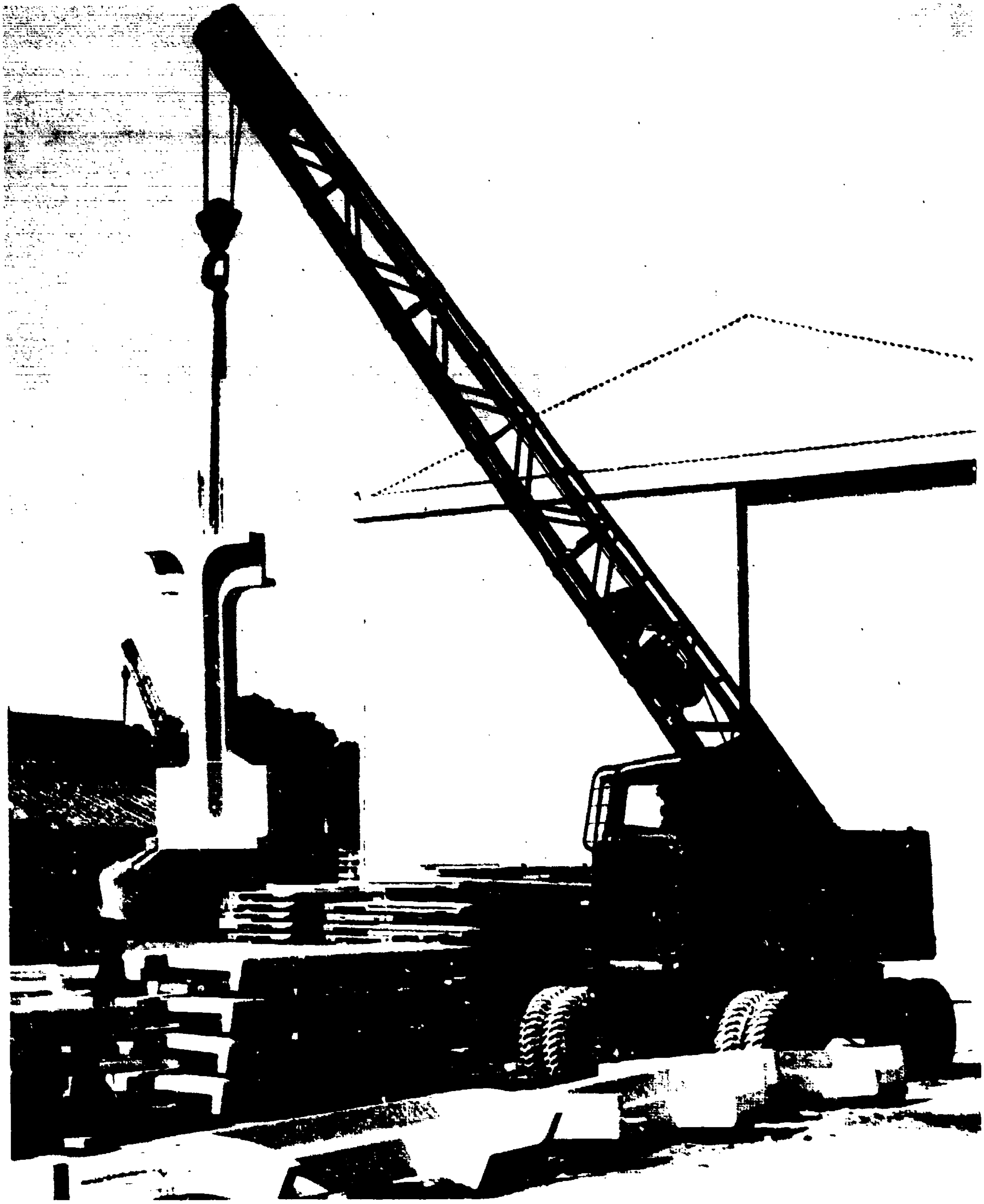

Figure C18. Crane used for handling precast concrete elements (Reprinted from "Precast Concrete Production" by J. G. Richardson)

APPENDIX D: NOZZLE OPERATOR'S DUTIES

The nozzleman's duties are to:

a. Ensure that all surfaces to be surfaced are clean and free of laitance or loose material, using air and air-and-water blast from the nozzle as required.

b. Ensure that the operating air pressure is uniform and provides proper nozzle velocity for good compaction.

c. Regulate the water content so that the mix will be plastic enough to give good compaction and a low percentage of rebound, but stiff enough not to sag. (In the dry mix process the nozzleman actually controls the mixing water, while in the wet mix process he directs changes in consistency as required.)

d. Hold the nozzle at the proper distance and as nearly normal to the surface as the type of work will permit to assure maximum compaction with minimum rebound.

e. Follow a sequence routine that will fill corners with sound shotcrete and encase reinforcement without porous material behind the steel using the maximum practicable layer thickness.

f. Determine necessary operating procedures for placement in close quarters, extended distances or around unusual obstructions where placement velocities and mix consistency must be adjusted.

g. Direct the crew when to start and stop the flow of material, and stop the work when material is not arriving uniformly at the nozzle.

h. Ensure that sand or slough pockets are cut out for replacement.

i. Bring the shotcrete to finished lines in a neat and workman-like manner.

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